THE FEASIBILITY OF IMPLEMENTING STEP FEED CONTROL OF STORM FLOW AT SELECTED WATER POLLUTION CONTROL PLANTS: PHASE I - PRELIMINARY INVESTIGATION

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Report Prepared For:

The Ontario Ministry of the Environment
Water Resources Branch

and

Environment Canada
Conservation and Protection
Wastewater Technology Centre

Report Prepared By:

D.J. Thompson, J.P. Bell and J. Kemp

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DISCLAIMER

This report has been reviewed by representatives from the Ontario Ministry of the Environment and the Wastewater Technology Centre on behalf of Environment Canada and approved for release to the public. The views expressed in the report are those of the authors and do not necessarily represent those of the Ontario Ministry of the Environment or Environment Canada.

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EXECUTIVE SUMMARY

Step feed control of storm flow has been successfully demonstrated at pilot scale and at the Dundas Water Pollution Control Plant (WPCP). Step feed will be considered as a control strategy in a position paper on Combined Sewer Overflows (CSO) being developed by the Ontario Ministry of Environment (MOE). The purpose of this project is to provide technical and cost information about step feed to support development of the CSO position paper.

The project will be carried out in two phases. This report describes Phase I of the project which includes:

1. A description of the step feed process and its potential for reducing storm flow impact.

2. A summary of the results of the demonstration project at the Dundas WPCP.

3. A preliminary evaluation of the feasibility of implementing step feed control of storm flow at five municipal WPCPs selected by the MOE.

4. General guidelines for estimating the storm flow capacity of wastewater treatment plants using step feed operation.

5. Estimates of the capital costs of retrofitting the five plants for step feed operation.

6. General guidelines for estimating the cost of modifying wastewater treatment plants for step feed operation.

A more comprehensive investigation of the technology and costs of step feed control of storm flow will be carried out in Phase II of the project.

Estimated storm flow capacities of the five plants using step feed ranged from 2.7 - 4.0 times design average flow capacity under average sludge settling conditions. The average estimated step feed flow capacity of the five plants was 3.2 times average design flow capacity.

The estimated cost ranges for modifying the plants for step feed operation were:

Substantial modifications required: 3400 - 6700 [$/(10^3 m^3/d)]

Moderate modifications required: 1000 - 4300 [$/(10^3 m^3/d)]

Minimal modifications required: 0 - 130 [$/(10^3 m^3/d)]

1. INTRODUCTION

Step feed control of storm flow has been successfully demonstrated at pilot scale and at the Dundas Water Pollution Control Plant (WPCP). Step feed will be considered as a control strategy in a position paper on Combined Sewer Overflows (CSO) being developed by the Ontario Ministry of Environment (MOE). The purpose of this project is to provide technical and cost information about step feed to support development of the CSO position paper.

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A more comprehensive investigation of the technology and costs of step feed control of storm flow will be carried out in Phase II of the project.

The plants selected for study in Phase I were:

-St. Catherines Port Dalhousie WPCP
-St. Catherines Port Weller WPCP
-Metropolitan Toronto Humber North WPCP
-Metropolitan Toronto Main WPCP
-Hamilton Woodward Avenue North WPCP

The historical flowrate characteristics including average dry weather flowrates, peak flowrates and bypass flowrates were collected for each plant. For each plant, the potential storm flow capacity under conventional operation was estimated from measurements of the mixed liquor settling characteristics and secondary settler surface area using the approach developed by Riddell et al. (1983). A mathematical model of the step feed process was utilized to determine the mixed liquor solids distribution under step feed operation. Finally, the storm flow capacity of each plant under step feed operation was determined using the predicted solids distribution.

The required physical modifications to allow for step feed operation were determined at each plant. The capital costs for these physical modifications were estimated. The capital cost estimates were also expressed as a cost per unit design flowrate. Three unit cost ranges are presented to represent three modification categories:

-minimal modifications required
-moderate modifications required
-substantial modifications required

These unit cost estimates will be utilized by the MOE to make a preliminary estimate of the cost of the retrofit to step feed on a province wide basis. Guidelines for application of the unit cost ranges are provided. Because of the time constraints and the limited number of plants studied, the unit cost estimates provided in Phase I should be regarded as preliminary.

2. DEVELOPMENT OF THE STEP FEED CONTROL STRATEGY

2.1 Fundamentals of Step Feed Operation

Municipal wastewater treatment facilities are frequently subject to rapid and sustained increases in flow rate caused by the entry of storm water into the sewer system. Because conventional activated sludge plants have limited hydraulic dampening capacity, high flow rates transfer additional solids from the aeration basin to the secondary settler and increase the possibility of solids washout. To avert washout many plants are forced to bypass the aeration tank and secondary settlers during high flow periods. Providing activated sludge plants with step feed capabilities can assist operators to prevent solids washout and reduce the frequency of bypass caused by peak flows.

A typical schematic for a plant with step feed capabilities is shown in Figure 1. The aeration basin is divided into four passes and the inlet channel equipped with gates or valves which allow influent wastewater to be added to one or more of the passes. Since 1939, when it was first introduced by Richard Gould at the Tallmans Island plant in New York City, step feeding has been employed at a number of activated sludge plants. Treatment plants in Richmond (Virginia), Portland (Maine), Phoenix (Arizona), New York City, Elkhart (Indiana) and Houston (Texas) employ step feeding.

By manipulating the point of influent wastewater addition, an operator can control the solids distribution within the aeration basin and reduce solids loading to the final settler. Figure 2, shows the effect of changing the point of addition to the middle and final passes of a three pass aeration basin. Adding influent to the second pass reduces the MLSS concentration in the final pass from 2500 to 1875 mg/L and the recycle sludge concentration from 5000 to 3750 mg/L. The concentration in the first pass increases from 2500 to 3750 mg/L. If all the influent wastewater is added to the final pass, the MLSS concentration in this pass decreases to 1500 mg/L and the recycle sludge concentration to 3000 mg/L. The net result of step feeding is to decrease the solids loading to the final settler.

As shown in Figure 3, the most severe step feed action (adding all of the influent wastewater to the last pass of the three passes) changes a conventional activated sludge process to the contact stabilization process. The last pass provides a short aeration period during which substrate is rapidly transferred from the wastewater to the mixed liquor. Upstream of the point of addition, the first two passes become a sludge reaeration zone in which substrate stored by the sludge is metabolized. Because most of the sludge is stored separately and cannot be washed out of the process during storm flows, contact stabilization plants are less susceptible to hydraulic washout from storm flows than are conventional plug flow plants. In addition, depending on the nature of the influent, contact stabilization plants are also better able to handle organic shock loads.

Instead of adding all of the influent wastewater to a single pass, influent can be simultaneously added to two or more passes during step feed operation (Figure 4). When the influent wastewater is divided equally between all passes, as shown in the middle schematic of Figure 4, the plant is operated in the step aeration mode. In comparison to the conventional plug flow plant, step aeration plants have more uniform oxygen requirements

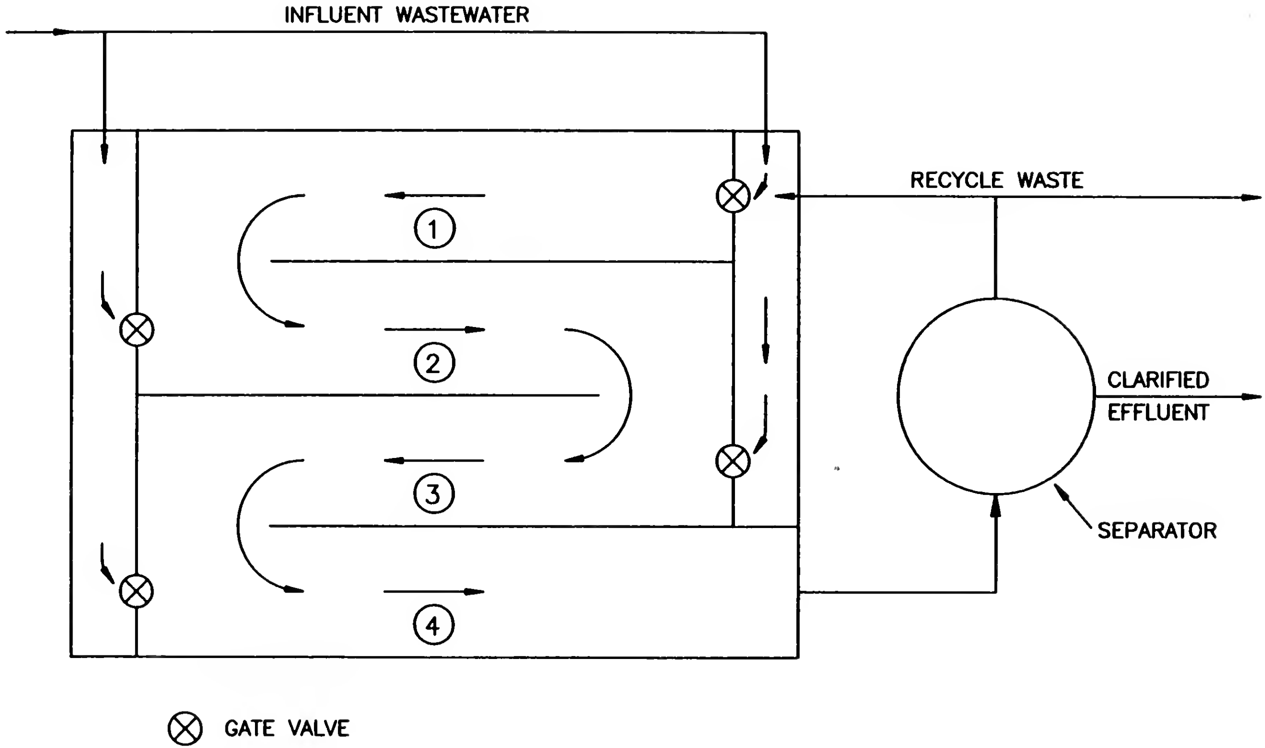

Figure 1 – Typical Step Feed Configuration

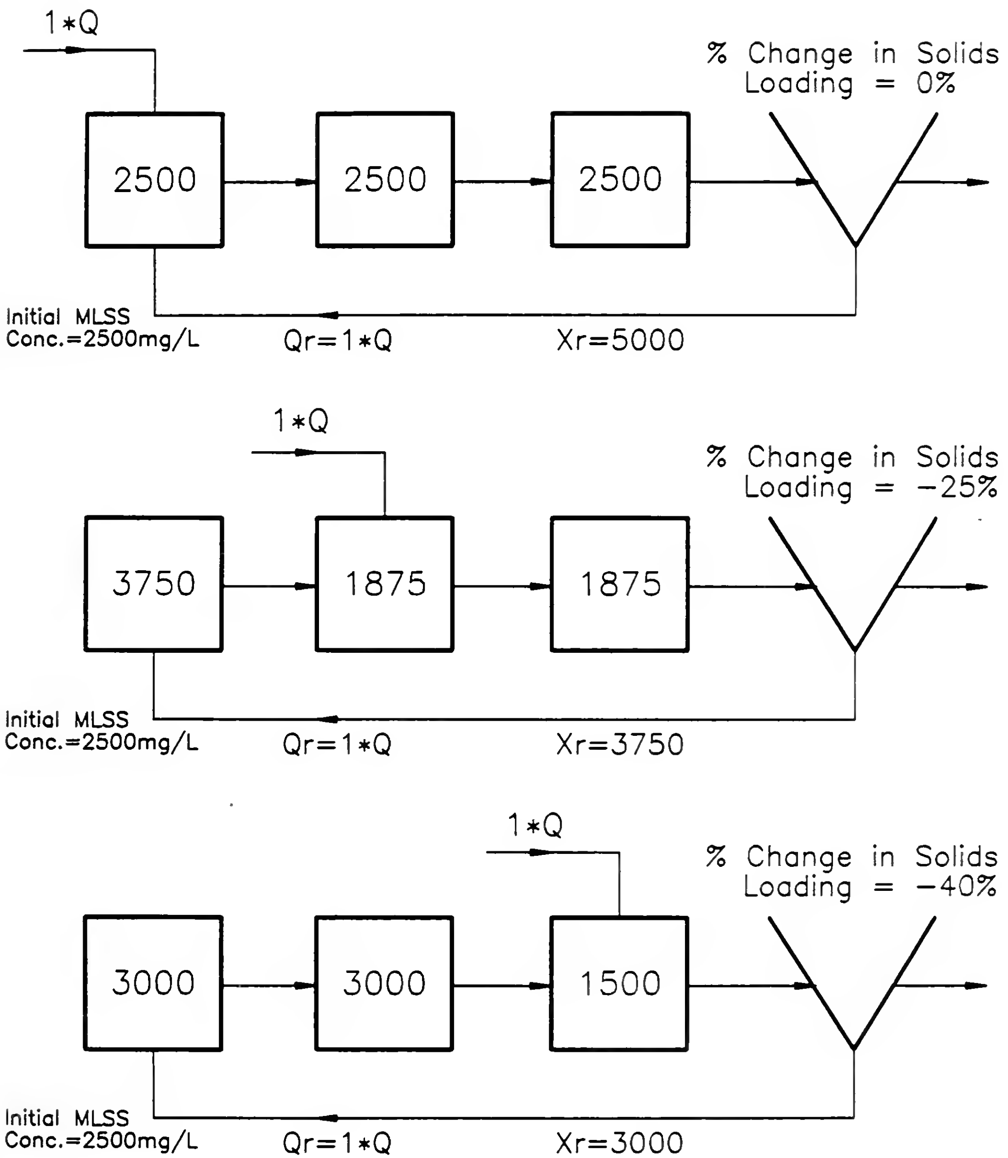

Figure 2 – Effect of Step Feed Position on MLSS Distribution

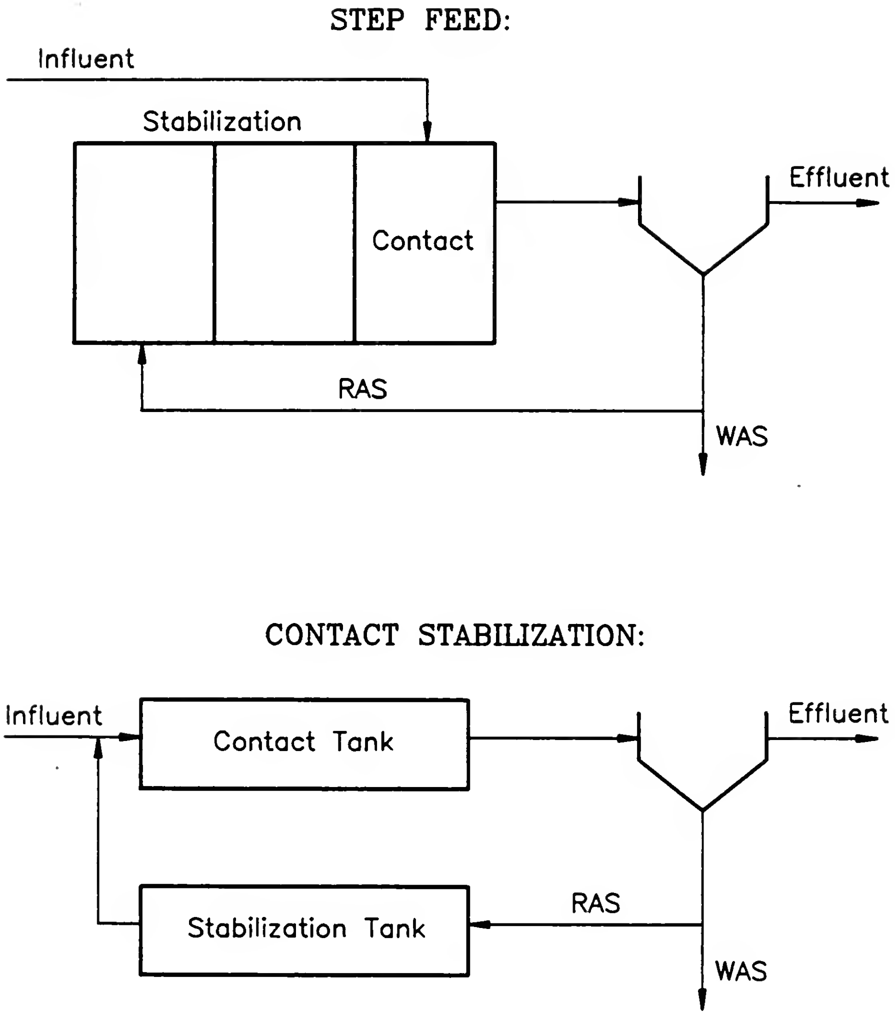

Figure 3 – Parallel Between Step Feed and Contact Stabilization

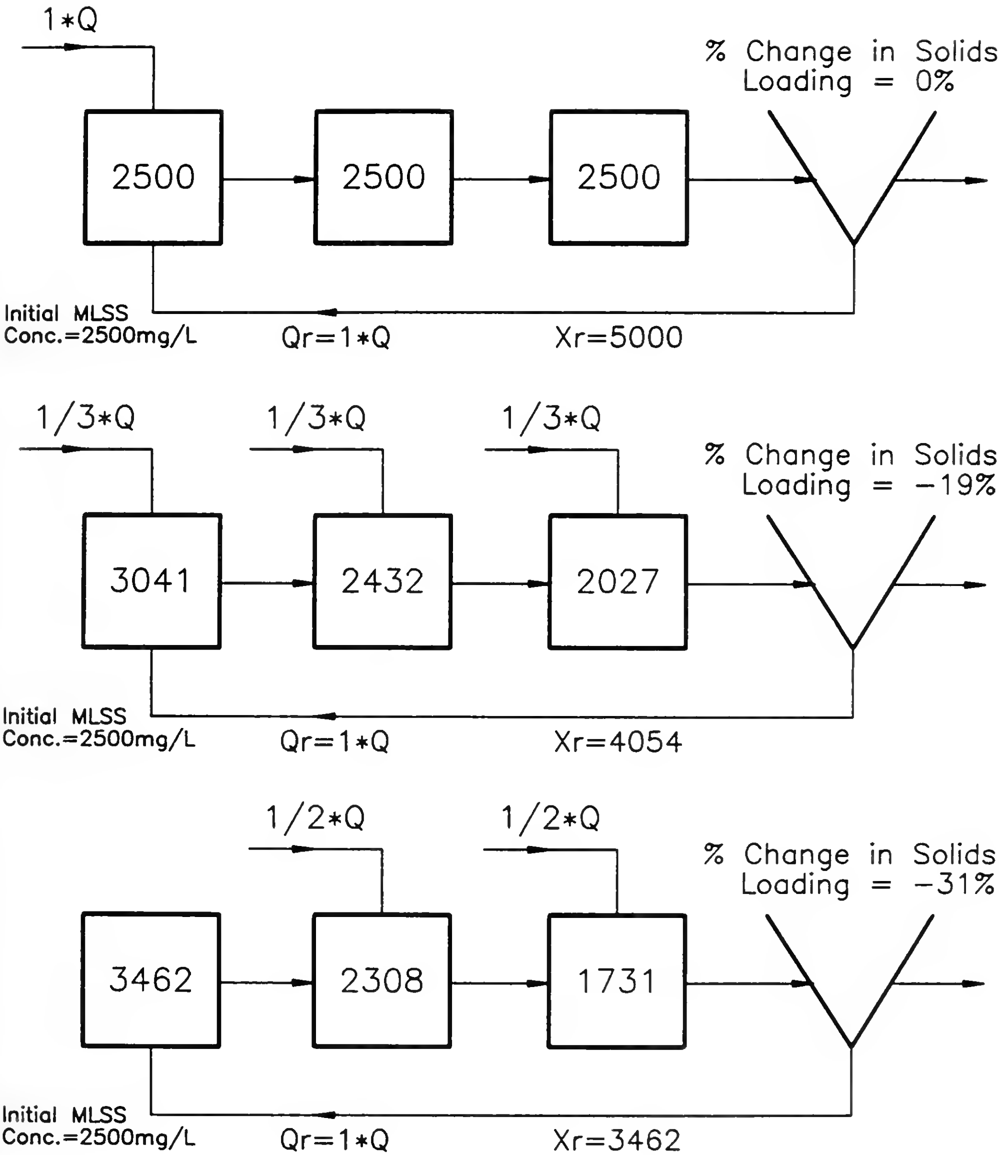

Figure 4 – Effect of Step Feed Position on MLSS Distribution

along the length of the basin and are less affected by hydraulic surges and organic shock loads.

In addition to the point or points of influent addition, other parameters which determine solids distribution in the aeration basin during step feeding include the sludge recycle rate, the wastewater influent flow rate, and the number of passes in the basin. As shown in Figure 5, increasing the recycle rate under step feeding increases the solids concentration in the last pass, the solids loading to the final settler, and hence decreases the effectiveness of step feeding. Therefore, operators using step feed operation during storm flows should not increase recycle rate, an approach commonly adopted during operation of conventional plants. Figure 6 shows that as the influent flow rate increases, the MLSS concentration in the last pass decreases further. Finally, as the number of passes in the aeration compartment increases, so does the effectiveness of step feeding in reducing final settler solids loading (Figure 7).

A number of factors should be considered in providing a conventional activated sludge plant with step feed capabilities. Hydraulic limitations on feed channels to the aeration basin should be checked under different wastewater flows and operational modes. Because step feeding decreases the solids concentration in the underflow from the final clarifiers, the capacity of the sludge wastage pumps should be checked to determine if there are any volumetric restrictions on sludge wastage. For basins without individual passes, baffles will be required. Options include curtain walls, concrete load-bearing walls or wooden baffles constructed of redwood and marine plywood. Step feed operation may require adjustment of air distribution or, possibly, the installation of additional aeration equipment. This can be determined by monitoring dissolved oxygen concentrations and effluent quality during step feed operation.

Because it increases the operational flexibility of a plant, step feed should be included in the design of all new activated sludge plants. Secondly, for existing plants which experience temporary or seasonal hydraulic overloading, consideration should be given to converting the plant to step feed operation. Conversion to step feed operation is likely less expensive than upgrading a plant by adding aeration basins or final clarifiers. Finally, plant operators should be trained in the use of step feed to minimize the effects of hydraulic overload.

For additional information concerning step feed operation, the paper by Buhr et al. (1984) is particularly recommended. The results of pilot-scale research on step feed operation carried out at the Wastewater Technology Centre are presented in the paper by Thompson et al. (1989).

2.2 Experimental Investigation of Step Feed Control

2.2.1 Pilot Plant Investigation

A pilot scale investigation of step feed control of storm flow was carried out at Environment Canada's Wastewater Technology Centre in 1988. The pilot plant consisted of three 22 m^3 aeration tanks in series feeding a circular secondary settler (Figure 8). The plant received a continuous supply of degritted wastewater from the Burlington Skyway WPCP.

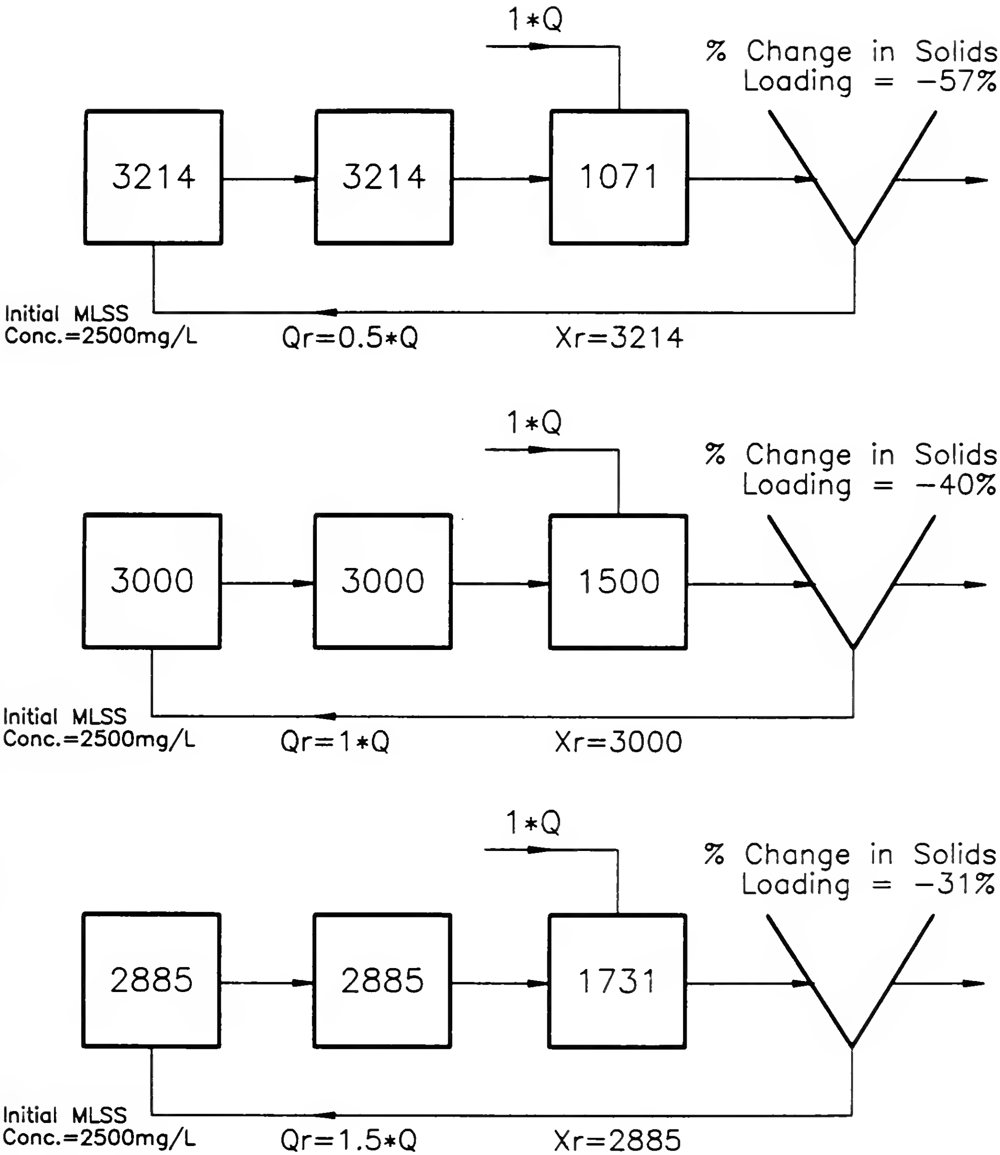

Figure 5 – Effect of Recycle Rate on MLSS Distribution

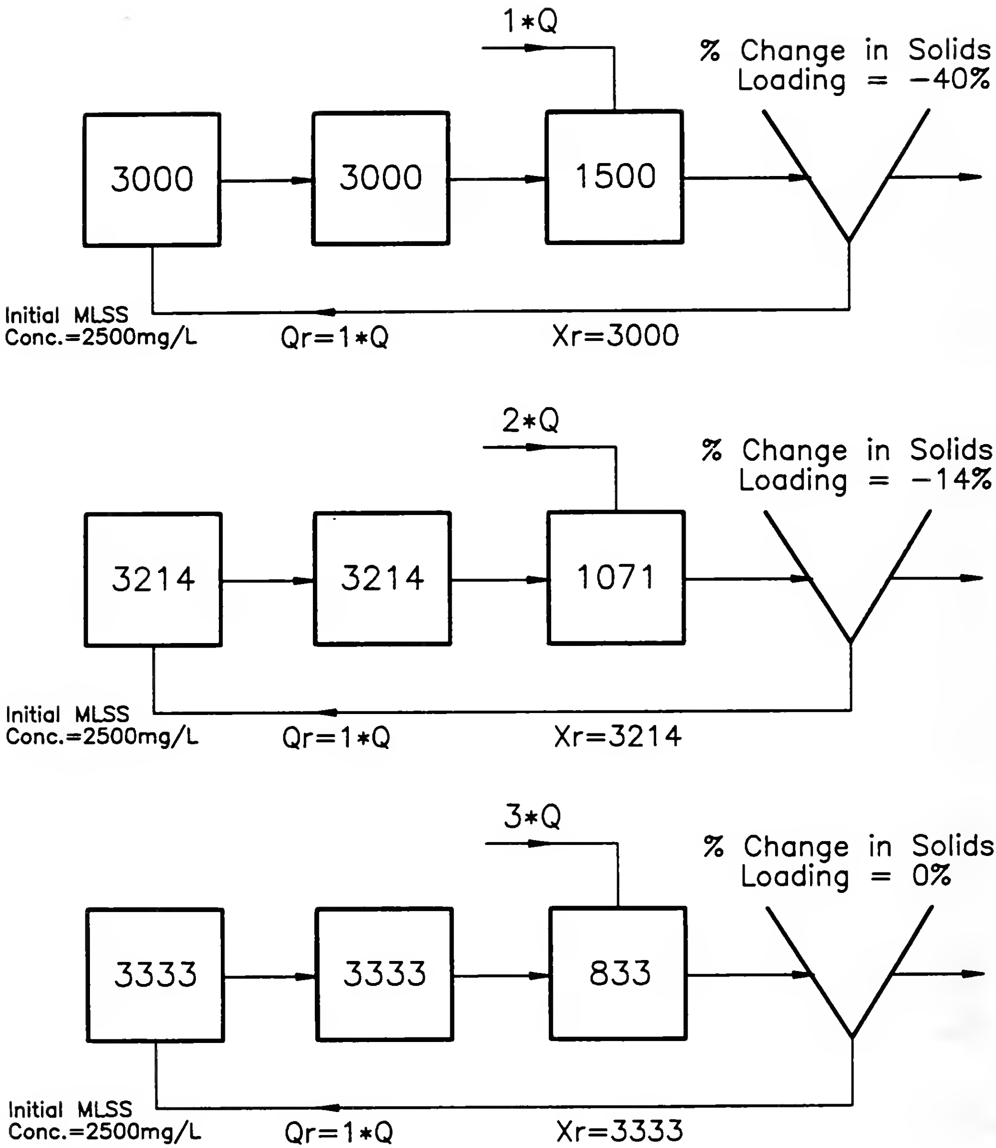

Figure 6 – Effect of Inflow on MLSS Distribution

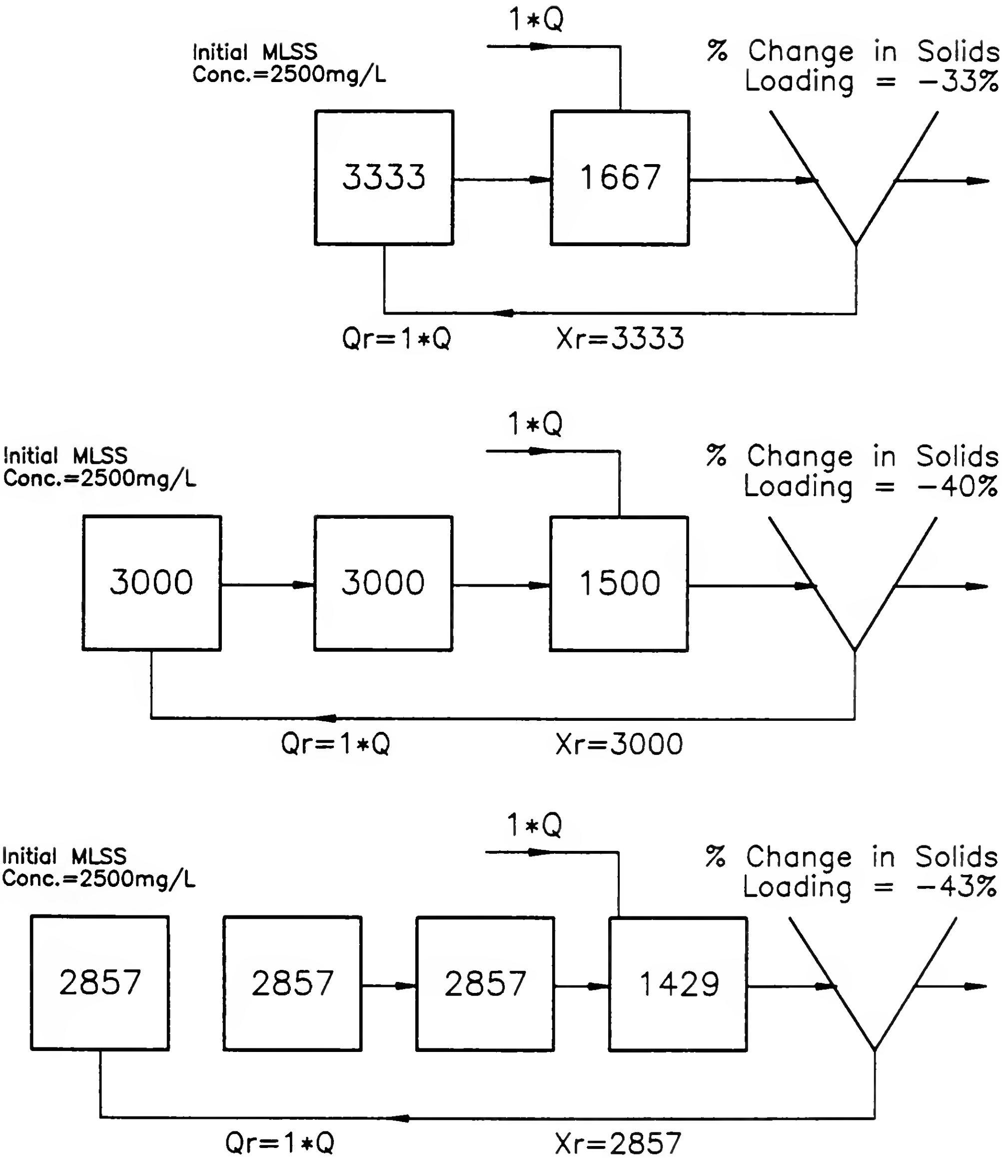

Figure 7 – Effect of Number of Tanks in Series on MLSS Distribution

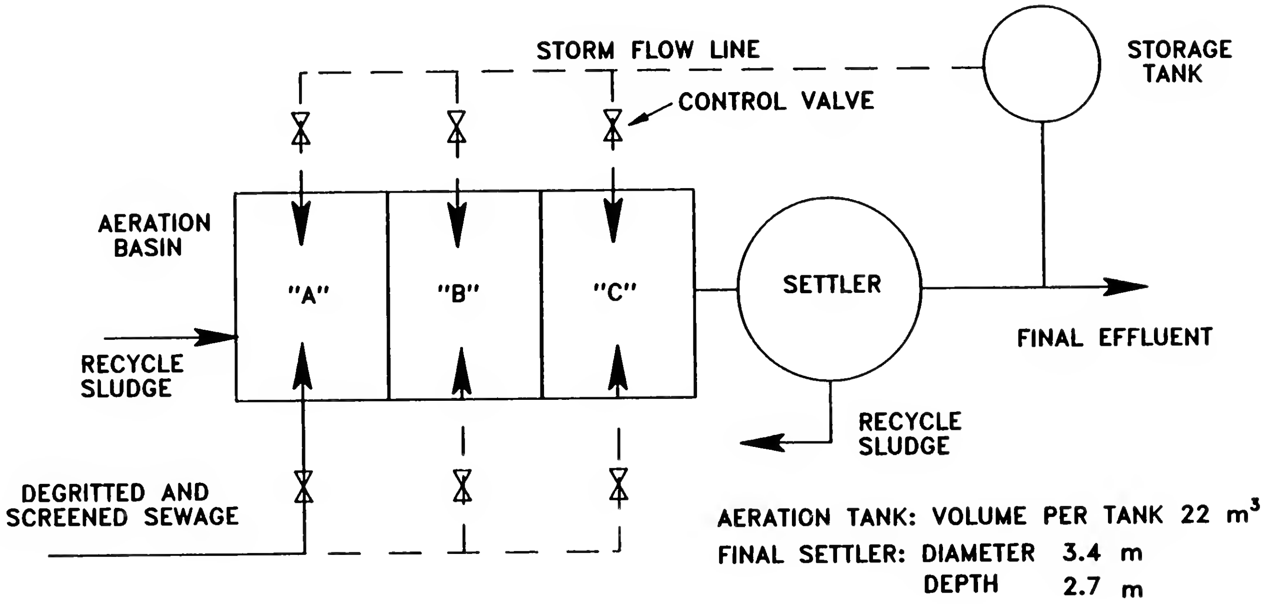

Figure 8– Pilot Plant Experimental Facility

The plant could be operated conventionally with wastewater directed into the first aeration tank or under step feed with wastewater entering either the second or third aeration tanks.

The pilot plant was operated conventionally and under step feed for extended time periods. As predicted, following the initiation of step feed operation there was a re-distribution of solids in the aeration tanks reducing the solids load to the secondary settler. The time to achieve the re-distribution of solids was approximately six hours. Under conventional operation the effluent suspended solids and BOD_5 concentrations were consistently below 10 mg/L. Under step feed there was a moderate decrease in effluent quality with effluent BOD_5 and suspended solids concentrations occasionally approaching 25 mg/L. Effluent quality decreased as the severity of the step feed action increased (feed directed to basin C rather than B) and the influent organic load increased.

Storm flows, up to 2.5 times the dry weather flowrate, were created by storing and recycling plant effluent to the aeration tanks. Without step feed control, these storm flows caused a rapid transfer of solids from the aeration tank to the secondary settler increasing the sludge blanket to the effluent weir level and causing massively high effluent suspended solids concentrations (Figure 9). When step feed was implemented at the same time as the storm flow entered the plant, there was a temporary transfer of solids from the aeration basin to the settler increasing the settler sludge blanket height. However, as step feed reduced the solids loading to the settler there was a reduction in sludge blanket height to a stable operating level and the effluent suspended solids concentration remained below 20 mg/L.

More detailed results from the pilot scale study are available from Thompson et al. (1989). The study concluded that step feed is a potentially effective method of increasing a water pollution control plant's hydraulic capacity for the control of storm flows. It was also concluded that a demonstration of step feed control be carried out at a full scale plant.

2.2.2 Full Scale Demonstration at the Dundas WPCP

The Dundas WPCP is a 18,000 m^3/day design flow municipal plant operated by the Regional Municipality of Hamilton Wentworth. The demonstration of step feed control was carried out at the Dundas WPCP by Enviromega in 1990. The Dundas WPCP actually consists of two separate plants, A and B (Figure 10). Step feed was carried out on Plant A only, allowing for a side by side comparison of performance. The entire plant flow could also be diverted to Plant A to simulate storm flow conditions. At plant A, step feed was initiated by the manipulation of two gates (Figure 11), and step feed was readily understood and implemented by the plant operating staff.

Plant A was operated conventionally and under step feed for a number of months. Effluent quality under conventional operation and step feed was very similar with effluent suspended solids and BOD_5 concentrations consistently below 10 mg/L. The plant achieved complete nitrification under conventional operation. With step feed, there was some bleed through of ammonia but effluent TKN concentrations remained below 2 mg/L.

The plant received a number of storm flows during the study period increasing the flow as high as three times the peak dry weather flowrate. The influent (potential bypass)

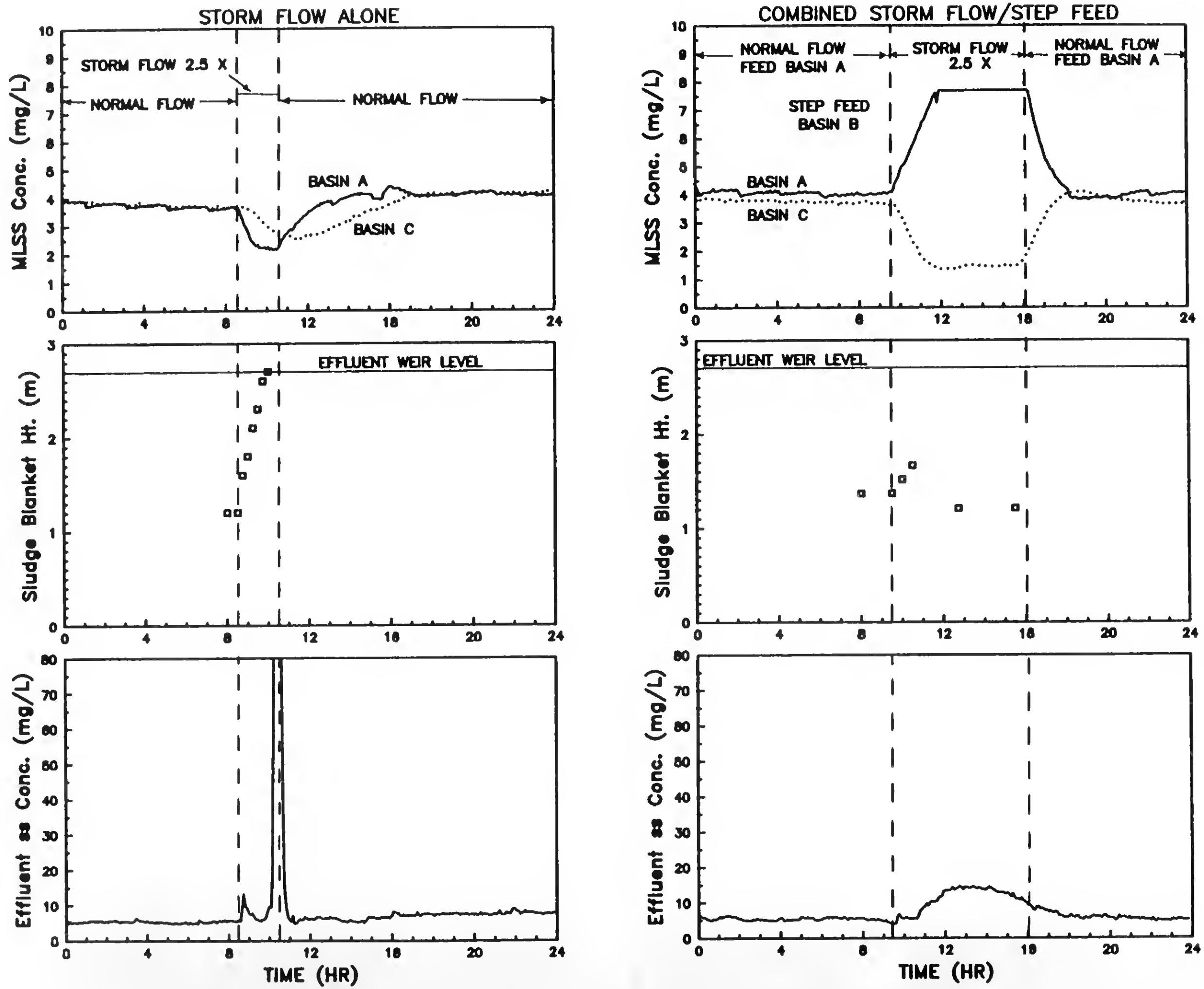

Figure 9 – MLSS Distribution, Sludge Blanket Height and Effluent Suspended Solids Concentration During Storm Flow and Combined Storm Flow/Step Feed

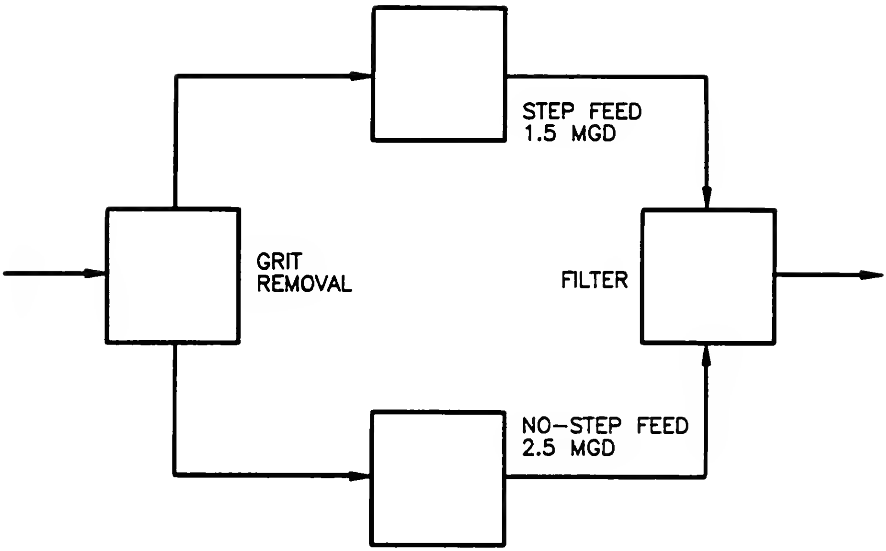

Figure 10 – Dundas WPCP Simplified Schematic

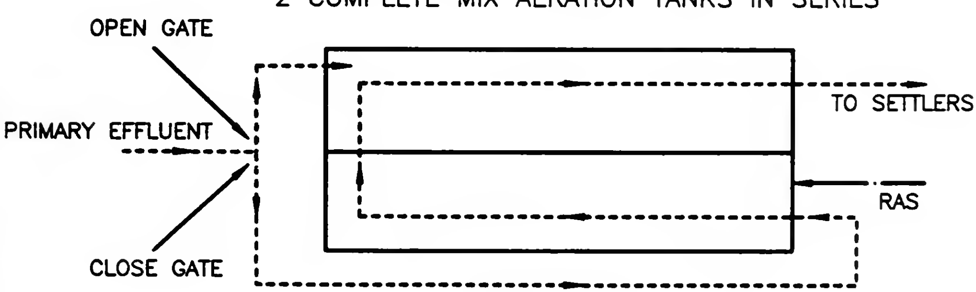

Figure 11 – Implementing Step Feed at the Dundas WPCP

during these storm flows was characterized by relatively low BOD_5 concentrations but very high suspended solids concentrations (> 200 mg/L). The primary settlers were not effective at substantially removing these solids because of the very high flowrate.

The plant performance during step feed control of storm flow was very similar to that observed at the pilot scale. The study confirmed that step feed is an effective means of avoiding plant washout during storm flows and may reduce the frequency of bypass at water pollution control plants. It was further concluded that step feed should be considered in the design and operation of all municipal water pollution control plants and that operators should be trained in the use of step feed. The complete study results will be available in the project report to be completed in March 1991.

3. ESTIMATING STORM FLOW CAPACITY

3.1 Basis of Estimate

The storm flow capacity of water pollution control plants is generally limited by the ability of the secondary clarifiers to separate the mixed liquor from the clarified effluent. Plants may operate relatively efficiently for many days while other long term operating and design criterion such as the ideal aeration basin hydraulic retention time (HRT) are exceeded because of the high flows. Most likely, plants will not achieve the effluent quality that they achieve under dry weather conditions, but as long as the secondary settlers continue to separate the mixed liquor from the effluent the plant will continue to achieve a significant reduction in the pollutant strength discharged to the receiving waters.

An approach for the design of secondary settlers presented by Riddell et al. (1983) was used as the basis for estimating secondary settler capacity (and thus total plant capacity). The approach can be briefly summarized as:

if: sludge ISV > SF * SOR (Equation 3.1a)
settlers operated below capacity

or if: sludge ISV < SF * SOR (Equation 3.1b)
settlers operated above capacity

where:

sludge ISV = initial settling velocity of mixed liquor entering settler

SOR = settler overflow rate (total plant flowrate divided by settler surface area)

SF = safety factor which is plant specific. For design purposes Riddell et al., (1983) recommend a SF ranging from one to three.

Thus, the plant capacity increases directly with the mixed liquor settling velocity and the settler surface area. It should be noted that the approach developed by Riddell et al. (1983) deals primarily with the thickening function of the secondary settlers. That is, it considers only the separation of the effluent from the thickened mixed liquor contained in the settler sludge blanket. It does not consider the required settler surface area or volume required for the production of a high quality effluent with low effluent suspended solids concentrations.

3.1.1 Effect of Step Feed on Storm Flow Capacity

Sludge initial settling velocity generally increases as the concentration of the sludge decreases. The relationship between sludge settling velocity and solids concentration has been expressed as a linear function, power function and exponential function:

ISV = b - a * X   linear   (Equation 3.2a)

$ISV = a' * X^{-n}$ power Equation 3.2b

$ISV = a'' * e^{-n'X}$ exponential Equation 3.2c

where:

X = solids concentration
a, a', a", b, n, n' = positive constants, characteristic of solids settling

For a given sludge, the most appropriate mathematical description for the relationship between sludge settling velocity and solids concentration can be determined experimentally. When predicting over a narrow range of solids concentrations, the linear model can be used as an approximation of the power and exponential models.

As explained in Section 2., the net effect of step feed operation is a reduction in the concentration of the mixed liquor fed to the secondary settlers. There are many mathematical models available for predicting the solids distribution in an aeration basin operated under step feed. The simplified model of Buhr et al. (1984) is rigorous yet mathematically simple. The required inputs to the models are the aeration basin hydraulic characteristics (e.g. number of tanks in series), the initial mixed liquor concentration the point of feed addition and the recycle flowrate. By coupling this step feed model with the model of the sludge settling characteristics (Equation 3.2a, 3.2b or 3.2c) the initial settling velocity of the mixed liquor entering the secondary settler can be determined for any step feed operating mode.

Knowing the sludge settling velocity for various operating modes, plant capacity can be determined by applying the relationships described by Equations 3.1a and 3.1b. As the point of feed addition is moved down the aeration basin, the concentration of the solids entering the secondary settler decreases, the sludge settling velocity of the solids entering the secondary settler increases, and the plant capacity increases.

There are limitations to the ability of step feed to increase plant capacity. As the point of influent feed addition is moved down the aeration basin, the contact time between influent wastewater and biomass is reduced. At some point, the contact time will become so short that little pollutant uptake occurs and bypass is essentially taking place. Determining the point beyond which little pollutant removal is occurring is difficult. However, a simple step feed strategy in which all feed is directed into the second pass of a two pass system was utilized successfully at the Dundas WPCP. If typical dry weather recycle flowrates are maintained, such an action substantially reduces the solids loading to the secondary settlers under storm flow. More severe step feed action is not recommended unless careful effluent quality monitoring is employed.

3.2 Description of Case Study WPCPs

One day site visits were made to each of the 5 case study plants during the week of December 10, 1990. The general process configuration was examined and dimensions of individual process units and historical flow data were obtained.

3.2.1 General Descriptions

Port Dalhousie WPCP:

The Port Dalhousie WPCP is a conventional activated sludge plant operated by the Regional Municipality of Niagara. The average dry weather flowrate to the plant is approximately 48,000 m^3/day. The treatment sequence consists of primary sedimentation, aeration and secondary clarification. An automatic bypass weir is located before the primary settlers. The bypass flowrate has been measured since December 11, 1989.

The plant contains a newly constructed gravity sedimentation storm tank. At high flows a portion of the influent wastewater will be diverted to the storm tank where it will be dosed with ferrous chloride. The overflow from the storm tank will be discharged to the receiving waters. The aeration basins consist of two complete mix tanks operated in parallel. The sizes of the main process units at the Port Dalhousie WPCP are summarized in Table 3.1.

Port Weller WPCP:

The Port Weller WPCP is a conventional activated sludge plant operated by the Regional Municipality of Niagara. The average dry weather flowrate to the plant is approximately 54,000 m^3/day. The treatment sequence consists of primary sedimentation, aeration and secondary clarification. An automatic bypass weir is located before the primary settlers. The bypass flowrate has been measured since March 8, 1990. A bypass following the primary settlers has recently been installed.

A polymer addition unit upstream of the primary settlers is presently being installed to be utilized during storm flow periods. Primary influent will also be dosed with ferrous chloride during high flow periods. The aeration system consists of two complete mix basins operated in parallel. The sizes of the main process units at the Port Weller WPCP are summarized in Table 3.1.

Toronto Humber WPCP:

The Toronto Humber WPCP is a conventional activated sludge plant operated by the Regional Municipality of Metropolitan Toronto. The average dry weather flowrate is approximately 416,000 m^3/day. The plant essentially consists of two separate plants both utilizing primary and secondary treatment. The North plant is presently being expanded. Approximately 66% of the eventual North plant is in operation. The South plant is designed to treat 63% of the total influent flow. The complete North plant is designed to receive approximately 37% of the influent flowrate. Automatic bypass weirs are located following the primary settlers in both the South and North plants and at the plant headworks. The total combined bypass flow is obtained by difference of various plant flow measurements.

The South plant aeration system consist of 5 basins each divided into 3 passes in series. The valves to the three passes are set to achieve an approximate 25%, 50%, 25% influent flow split between passes 1, 2 and 3. During storm flows the influent gates to all

Table 3.1- Case Study WPCPs; Size of Main Process Units

Port Dalhousie:

	Number of Tanks	Tank Dimensions (L X W X D); meters
Primary Settlers	2	49 X 15 X 3.0
Storm Tanks	1	45 X 11 X 3.6
Aeration Tanks	2	73 X 18 X 5
Secondary Settlers		
-rectangular	4	67 X 12 X 3.6
-square	1	39 X 39 X 3.6

Port Weller:

	Number of Tanks	Tank Dimensions (L X W X D); meters
Primary Settlers	2	47 X 15 X 3.0
Aeration Tanks	2	61 X 15 X 4.7
Secondary Settlers		
-square	4	20 X 20 X 3.0
-circular	2	33 m diam, 4.5 m deep

Humber South:

	Number of Tanks	Tank Dimensions (L X W X D); meters
Primary Settlers	6	72 X 10 X 3.0
Aeration Tanks	5	139 X 8 X 4.6
Secondary Settlers		
-square	12	29 X 29 X 3.9
-circular	3	48 m diam, 4.6 m deep

continued

Table 3.1 Continued

	Number of Tanks	Tank Dimensions (L X W X D); meters
Humber North:		
Primary Settlers	3	79 X 25 X 3.6
Aeration Tanks	3	73 X 18 X 7.5
Secondary Settlers	6	48 m diam, 4.6 m deep

Main WPCP:

	Number of Tanks	Tank Dimensions (L X W X D); meters
Primary Settlers		
-group 1	6	61 X 20 X 4.5
-group 2	3	76 X 32 X 4.7
-group 3	3	97 X 36 X 4.8
Aeration Tanks	9	162 X 24 X 4.6
Secondary Settlers	9	116 X 26 X 3.8
Woodward South:		
Secondary Settlers	4	73 X 18 X 3.7

Woodward North:

	Number of Tanks	Tank Dimensions
Secondary Settlers	8	37 X 37 X 3.0

three passes are opened wide. When completed, the North plant aeration system will consist of 3 complete mix aeration basins operated in parallel. The sizes of the main process units at the Humber WPCP are summarized in Table 3.1. For the purposes of this study only the North plant will be considered.

Toronto Main WPCP:

The Toronto Main WPCP is a conventional activated sludge plant operated by the Regional Municipality of Metropolitan Toronto. The average dry weather flowrate to the plant is approximately 774,000 m^3/day. The plant consists of primary sedimentation, aeration and secondary clarification. A bypass gate is located following one set of primary settlers. The bypass flow is obtained by difference of various plant flow measurements.

The aeration system consists of 9 aeration tanks each divided into 4 passes. The valves to the four passes are set to achieve an approximate 40%, 30%, 20%, 10% flow split between passes 1, 2, 3 and 4. The sizes of the main process units at the Main WPCP are summarized in Table 3.1.

Woodward Avenue WPCP:

The Woodward Avenue WPCP is a conventional activated sludge plant operated by the Regional Municipality of Hamilton Wentworth. The average dry weather flowrate to the plant is approximately 409,000 m^3/day. Ferrous chloride is added to the primary influent to reduce the organic load to the aeration basins and for phosphorous control. Following primary sedimentation the plant is split into two plants, North and South, both consisting of aeration and secondary clarification. A bypass gate is located following the primary settlers and at the plant headworks. Bypass flowrate is not presently measured.

The North plant receives approximately 66% of the influent flow and the South plant 34%. The North plant aeration system consists of eight basins operated in parallel. Each basin consists of six mechanically mixed cells partially separated by baffles. The North plant aeration system is presently being modified to allow for primary effluent to be step fed down the length of the aeration basins. The South plant aeration system consists of four complete mix basins operated in parallel. The sizes of the main process units at the Woodward Avenue WPCP are summarized in Table 3.1. For the purposes of this study only the North plant will be considered.

3.2.2 Flowrate Characteristics

Port Dalhousie WPCP:

The Port Dalhousie Plant Certificate of Approval lists the plant secondary treatment rated capacity as 61,350 m^3/day average daily flow and 100,000 m^3/day peak flow following the commissioning of the storm flow tank. The plant is also rated for an additional 22,700 m^3 of primary treatment with chemical precipitation using the recently constructed storm flow tank. Historically (and at the time of the site visit) the level of the bypass weir was set to treat (primary and secondary) approximately 127,232 m^3/day of wastewater before bypass occurred. The operators have not noticed imminent plant hydraulic limitations at this peak

flowrate.

The average flowrate to the plant (excluding bypass) for the period January 1990 to July 1990 (inclusive) was 48,000 m^3/day. The total bypass flow from December 11, 1989 to October 9, 1990 was 433,540 m^3. There were 52 bypass events during this period. Table 3.2 summarizes the Port Dalhousie flow data to be utilized for the remainder of this report.

Port Weller WPCP:

The Port Weller WPCP Certificate of Approval lists the plant secondary treatment rated capacity as 56,182 m^3/day following the commissioning of the coagulant dose system upstream of the primary settlers. It is assumed that this limit refers to the average daily flow. The plant primary treatment rated capacity is 136,200 m^3/day. It is assumed that this limit refers to peak flow. It is not clear what the secondary treatment peak flow rated capacity is. For purposes of this study it was assumed to be 112,364 m^3/day (twice the average flow rated capacity) although the plant superintendent believed it to be 56,182 m^3/day (no increase over average flow capacity). The plant operator is investigating this discrepancy. Historically (and at the time of the site visit) the level of the bypass weir was set to treat (primary and secondary) approximately 100,000 m^3/day. The operators have not noticed imminent plant hydraulic limitations at this peak flowrate.

The average flowrate to the plant (excluding bypass) for the period January 1990 to July 1990 (inclusive) was 53,684 m^3/day. The total bypass flow from March 8, 1990 to October 12, 1990 was 100,605 m^3. There were 19 bypass events during this period. Table 3.2 summarizes the Port Weller flow data to be utilized for the remainder of this report.

Toronto Humber WPCP:

The Humber WPCP Certificate of Approval was not available from plant staff. Upon completion of the North plant the total plant design average flowrate is 472,576 m^3/day. The peak design flowrate for primary treatment is 1,022,400 m^3/day. The peak design flowrate for secondary treatment is 945,152 m^3/day. For the North plant, the design average flowrate is 174,853 m^3/day and the peak design flowrate is 349,706 m^3/day. Operators advised that a second final effluent gate is opened during high flowrates.

The average flowrate to the plant (excluding bypass) for the period November 1989 to October 1990 (inclusive) was 416,000 m^3/day. The total recorded bypass (receiving primary treatment only) during this period was 1,430 m^3. Table 3.2 summarizes the Humber flow data to be utilized for the remainder of this report.

Toronto Main WPCP:

The Main WPCP Certificate of Approval was not available from plant staff. The design average flowrate to the plant is 817,920 m^3/day. The peak design flowrate to the plant is 1,635,840 m^3/day. In practice this peak flowrate is maintained for only 1.5 hours. The flow through the plant is returned to 817,920 m^3/day following this period.

The average flowrate to the plant (excluding bypass) for the period December 1989 to

November 1990 (inclusive) was 774,000 m^3/day. The total recorded bypass (primary treatment only) during this period was 115,000 m^3. Table 3.2 summarizes the Main WPCP flow data to be utilized for the remainder of this report.

Woodward Avenue WPCP North Plant:

Only the Woodward Avenue WPCP North Plant will be considered in this report. The Woodward Avenue WPCP Certificate of Approval was not obtained in sufficient time for this report. The design average flowrate to the North plant is 272,640 m^3/day. For purposes of this report, the design peak flowrate is assumed to be 545,280 m^3/day (twice the design average flowrate). The bypass flow is not yet recorded. Table 3.2 summarizes the Woodward Avenue North WPCP flow data to be utilized for the remainder of this report.

Table 3.2- Case Study Plants: Flow Characteristics

	Design Flow		Historical Avg. Flow (*)
	Avg.	Peak	
	(m^3/day)	(m^3/day)	(m^3/day)
Port Dalhousie	61,350	100,000	48,000
Port Weller	56,182	112,364	53,684
Humber	472,576	945,152	416,000
Humber North	174,853	349,706	NA
Main	817,920	1635,840	774,000
Woodward North	272,640	545,280	NA

(*)- excluding bypass flow

NA = not available

Note: Port Dalhousie and Port Weller design flows are based on the plant Certificate of Approval. Design flows for the other plants are based on discussions with the plant staff.

3.2.3 Secondary Settler Surface Area

Table 3.3 summarizes the total secondary settler surface area for each case study plant, the overflow rate at the design average flowrate and the design peak flowrate. The Port Dalhousie WPCP has the most relative secondary settler surface area while the Main

WPCP has the least. The Port Dalhousie overflow rate at the design peak flowrate is less than the Main flowrate at the design average flowrate.

Table 3.3- Case Study Plants: Available Secondary Settler Surface Area

	Total Surface Area	Overflow Rate	
		Design Flow	Peak Flow
	(m^2)	(m/h)	(m/h)
Port Dalhousie	4737	0.54	0.88
Port Weller	3309	0.71	1.41
Humber North	10851	0.67	1.34
Main	27144	1.26	2.51
Woodward Ave. N	10952	1.04	2.08

The MOE guideline for secondary settler peak overflow rate is 1.48 m/h (Ontario Ministry of the Environment, 1984). It is assumed that the limit refers to peak dry weather flowrate. If it is assumed that the peak dry weather flowrate is 1.5 times the average flowrate, the Main Plant and the Woodward Ave. North Plant exceed the recommended guideline. However, both of these plants have large pump station wet wells which allow flow equalization which possibly reduces the ratio of the peak to the average flow below 1.5.

3.2.4 Sludge Settling Characteristics

During each site visit three settling tests on the plant mixed liquor were performed. The first test was performed with the mixed liquor at full concentration. The second and third tests were performed with the mixed liquor diluted to 75% and 50 % of the original concentration using plant effluent. All tests were performed using the recommended procedure of White (1975) with a 3.5 L stirred vessel. The two parameters measured were the sludge initial settling velocity (ISV) and the stirred sludge volume index (SSVI). The results of the settling tests are presented in Table 3.4.

The plants with the highest reported SSVI were Port Dalhousie, Humber North and Woodward Avenue North. Chapman (1984) classified a sludge with an SSVI of 40 - 50 mL/g as good settling. Only the Toronto Main WPCP and Port Weller WPCP sludges fall into this category. The remaining sludges are classified as average settling (SSVI < 120 mL/g).

Table 3.4- Case Study Plants: Sludge Settling Characteristics Measured During Site Visits

Port Dalhousie:

SS Conc. (X) (g/L)	SSVI (mL/g)	ISV (m/h)	Linear Equation (3.2a)
2.79	65	2.64	ISV = -1.67*X + 7.18
1.82	65	3.81	
1.24	57	5.31	R^2= 0.95

*- the Port Dalhousie staff reported a mixed liquor SVI of 138 on the site visit day

Port Weller:

SS Conc. (X) (g/L)	SSVI (mL/g)	ISV (m/h)	Linear Equation (3.2a)
2.25	51	3.81	ISV = -3.51*X + 11.5
1.51	46	5.46	
1.22	46	7.75	R^2= 0.89

*- the Port Weller staff reported a mixed liquor SVI of 85 on the site visit day

Main:

SS Conc. (X) (g/L)	SSVI (mL/g)	ISV (m/h)	Linear Equation (3.2a)
2.82	44	4.28	ISV = -1.97*X +9.81
2.02	40	5.77	
1.36	38	7.16	R^2= 0.99

*- the Main staff reported a mixed liquor SVI of 61 on the site visit day

continued

Table 3.4 Continued

Humber North:

SS Conc. (X)	SSVI	ISV	Linear Equation (3.2a)
(g/L)	(mL/g)	(m/h)	
2.75	66	2.24	ISV = -1.41*X +6.01
2.17	63	2.75	
1.40	64	4.12	R^2= 0.97

-the test was performed using North plant mixed liquor

-the Humber plant staff reported a North plant mixed liquor SVI of 127 on the site visit day

Woodward Avenue North:

SS Conc. (X)	SSVI	ISV	Linear Equation (3.2a)
(g/L)	(mL/g)	(m/h)	
2.40	96	2.08	ISV = -1.74*X +6.15
1.72	87	2.93	
1.06	77	4.42	R^2= 0.97

For each plant, Equation 3.2a was used to correlate the sludge ISV to the solids concentration. The correlation results are also presented in Table 3.4. The minimum linear regression coefficient of determination was 0.89 suggesting an acceptable fit. The slopes of the individual equations ranged from 1.41 to 3.51 suggesting that a relatively small decrease in solids concentration could cause a large increase in sludge settling velocity.

For each plant the historical reported SVI results were obtained to indicate the historical variability in sludge settling characteristics. These data are presented in Table 3.5. The Port Dalhousie, Humber North and Woodward Ave. North plants displayed large variations in reported SVI values. It should be noted that on individual days SVI values were reported exceeding the monthly average maximum reported in Table 3.5. These plants appear to be highly subject to sludge bulking. For these plants the actual storm flow capacity

would also very considerably with time. The Port Weller and Main WPCPs reported much more consistent and lower SVI values.

Table 3.5- Case Study Plants: Historical Sludge Settling Characteristics

Port Dalhousie:

Site Visit			Historical	
SSVI	SVI	MLSS	SVI Range	MLSS Range
(mL/g)	(mL/g)	(g/L)	(mL/g)	(g/L)
65	138	2.8	99-283	1.6-3.1

- the historical data represent the range of average monthly SVI and MLSS concentration covering the period of January 1990 to July 1990 (inclusive)

Port Weller

Site Visit			Historical	
SSVI	SVI	MLSS	SVI Range	MLSS Range
(mL/g)	(mL/g)	(g/L)	(mL/g)	(g/L)
51	85	2.3	56-85	2.5-4.0

- the historical data represent the range of average monthly SVI and MLSS concentration covering the period of January 1990 to July 1990 (inclusive)

Main:

Site Visit			Historical	
SSVI	SVI	MLSS	SVI Range	MLSS Range
(mL/g)	(mL/g)	(g/L)	(mL/g)	(g/L)
44	61	2.8	37-79	2.9-5.5

- the historical data represent the range of average monthly SVI and MLSS concentration covering the period of December 1989 to November 1990 (inclusive)

continued

Table 3.5 Continued

Humber (average of South and North plants):

Site Visit			Historical	
SSVI (mL/g)	SVI (mL/g)	MLSS (g/L)	SVI Range (mL/g)	MLSS Range (g/L)
66	117	2.8	152-399	2.2-3.6

- the historical data represent the range of average monthly SVI and MLSS concentration covering the period of November 1989 to October 1990 (inclusive)

Woodward Ave. North:

Site Visit			Historical	
SSVI (mL/g)	SVI (mL/g)	MLSS (g/L)	SVI Range (mL/g)	MLSS Range (g/L)
96	154(*)	2.4	100-240	NA

- the historical data represent the range of average monthly SVI data measured by the Wastewater Technology Centre during a study covering the period of April 1989 to April 1990

NA= not available

*- assumed to equal 1.6 times SSVI

3.3 Storm Flow Estimation Procedure as Applied to Each Case Study Plant

3.3.1 Storm Flow Capacity with Conventional Operation

The storm flow capacity of each plant under conventional operation was calculated by applying Equation 3.1 which relates the settler hydraulic capacity to the initial settling velocity (ISV) of the sludge entering the settler. The following relationship between the ISV,

the sludge volume index (SVI), and the mixed liquor suspended solids (MLSS) concentration was suggested by Daigger and Roper (1985):

$$ISV = 7.80*\text{Exp}[-MLSS*(0.148 + 0.00210*SVI)] \qquad \text{Equation 3.3}$$

where:

ISV = initial settling velocity of mixed liquor, m/h,
MLSS = mixed liquor suspended solids concentration, g/L,
SVI = sludge volume index, mL/g.

To provide a range of settling conditions, Equation 3.3 was used to estimate the ISV for each plant under conventional operation at the following three conditions:

(1) The highest historical MLSS concentration and SVI
(2) The average historical MLSS concentration and SVI
(3) The lowest historical MLSS concentration and SVI

The MLSS concentration and SVI values used in this analysis were based on the historical monthly average values reported in Table 3.5.

The ISV estimates obtained from Equation 3.3 were used with Equation 3.1 to estimate the storm flow capacity of each plant under each of the three settling conditions considered. For each plant, a safety factor of 1.5 was applied in Equation 3.1 to account for non-ideal operating conditions such as settlers out of service or non-uniform hydraulic distribution between settlers. To maximize potential plant capacity, sludge settling velocity measurements should be made daily and the plant capacity estimated daily using equation 3.1. In addition, the ideal flow distribution between settlers should be maintained.

3.3.2 Storm Flow Capacity with Step Feed Operation

The storm flow capacity of each plant under step feed operation was also calculated using Equation 3.1. A safety factor of 1.5 was again uniformly applied in Equation 3.1. The ISV values were estimated as described in Section 3.3.1 using Equation 3.3. The three historical settling conditions described above were again considered.

For each plant, the aeration system was modelled by applying the Buhr model (section 3.1.1) using a Lotus 123 spreadsheet. The typical dry weather recycle flowrate was consistently maintained. An iterative procedure was utilized to estimate plant capacity under step feed operation. Initially, a moderate step feed action was selected. For a given influent flowrate (beginning at 1.25 times the maximum conventional capacity flowrate) the model of the aeration system predicted the solids concentration of the mixed liquor fed to the secondary settler. The settling velocity of the sludge entering the secondary settler was then calculated from the solids concentration using Equation 3.3. The settler overflow rate at this flowrate was then compared to the settling velocity of the mixed liquor entering the settler. If the settler overflow rate exceeded the sludge settling velocity the severity of the step feed action was increased. If the overflow rate did not exceed the capacity, the influent flowrate was increased and another iteration was performed. This process was continued until either

the contact time between influent wastewater and mixed liquor was less than one hour, or the mixed liquor concentration fed to the secondary settler was less than 1000 mg/L. These limits were selected to ensure a reasonable level of biological treatment and are based on previous experience with step feed operation. The selection of optimal limits to step feed operation requires experimentation at the plant. The somewhat arbitrary limits selected indicate the preliminary nature of the capacity estimates in this report.

3.3.3 Results

The estimated storm flow capacity of the case study plants under conventional and step feed operation are presented in Table 3.6. The capacity was estimated for the three settling conditions described in Section 3.3.1. The step feed operating conditions for each case are presented in Table 3.7. The plant capacity estimates presented here are based on historical operating practices. In the future, plants may be required to adopt biological nitrogen removal. This would require the use of higher MLSS concentrations which would reduce the estimated plant capacities under conventional and step feed operation. Also the potential bleed through of ammonia might limit the degree of step feeding which could be carried out.

The following are sample calculations of flow capacity for the Port Dalhousie WPCP to illustrate the estimation procedure:

<u>Port Dalhousie Plant Parameters</u>:

Plant secondary clarifier area surface (A) = 4737 m^2

Historical average MLSS concentration (MLSS) = 2.5 g/L

Historical average SVI (SVI) = 169

Safety factor (SF) = 1.5

<u>Conventional Operation</u>:

Using Equation 3.3,

Initial settling velocity (ISV) = 7.80*Exp[-MLSS*(0.148 + 0.00210*SVI)] =

7.80*Exp[-2.5*(0.148 + 0.00210*169)] = 2.219 m/h

Using Equation 3.1,

Maximum allowable settler overflow rate (SOR) = ISV/SF = 2.219/1.5 = 1.479 m/h

Calculating maximum allowable plant flow rate (Q_{max}),

Q_{max} = SOR*A*24 = 1.479*4737*24 = 168,000 m^3/d

Table 3.6– Case Study Plants: Estimated Storm Flow Capacity Under Conventional and Step Feed Operation

	Avg. MLSS, SVI		High MLSS, SVI		Low MLSS, SVI	
	Flow (1000 m^3/d)	% Design Avg.	Flow (1000 m^3/d)	% Design Avg.	Flow (1000 m^3/d)	% Design Avg.
Port Dalhousie:						
Conventional Capacity	168	273	59	96	334	544
Step Feed Capacity	245	399	185	301	334	544
Port Weller:						
Conventional Capacity	168	299	111	197	212	377
Step Feed Capacity	205	365	163	290	237	421
Main:						
Conventional Capacity	1338	163	603	74	1760	215
Step Feed Capacity	2176	266	1840	225	2370	289
Humber North:						
Conventional Capacity	254	145	38	22	502	287
Step Feed Capacity	482	276	355	203	651	372
Woodward Ave. North:						
Conventional Capacity	503	185	259	95	727	267
Step Feed Capacity	823	301	620	227	941	345

Table 3.7- Step Feed Operating Conditions

Port Dalhousie:

	Avg SVI, MLSS	High SVI, MLSS	Low SVI, MLSS
Average MLSS (g/L)	2.5	3.1	1.6
SVI (mL/g)	169	283	99
No. Tanks in Series	2	2	2
Flow Split (%)	41/59	15/85	100/0
Recycle Flow (1000 m^3/d)	69	52	94
Last Pass MLSS (g/L)	1.8	1.6	1.6
Contact Time (h)	1	1	1

Port Weller:

	Avg SVI, MLSS	High SVI, MLSS	Low SVI, MLSS
Average MLSS (g/L)	3.0	4.0	2.5
SVI (mL/g)	71	85	56
No. Tanks in Series	2	2	2
Flow Split (%)	50/50	60/40	37/63
Recycle Flow (1000 m^3/d)	82	95	65
Last Pass MLSS (g/L)	2.4	2.1	2.8
Contact Time (h)	1	1	1

Main:

	Avg SVI, MLSS	High SVI, MLSS	Low SVI, MLSS
Average MLSS (g/L)	3.5	5.5	2.9
SVI (mL/g)	57	79	37
No. Tanks in Series	4	4	4
Flow Split (%)	12/0/88/0	0/0/100/0	20/0/80/0
Recycle Flow (1000 m^3/d)	655	510	644
Last Pass MLSS (g/L)	1.4	2.0	1.6
Contact Time (h)	1	1	1

Humber North:

	Avg SVI, MLSS	High SVI, MLSS	Low SVI, MLSS
Average MLSS (g/L)	2.8	3.6	2.0
SVI (mL/g)	213	399	168
No. Tanks in Series	2	2	2
Flow Split (%)	28/72	0/100	46/54
Recycle Flow (1000 m^3/d)	145	107	195
Last Pass MLSS (g/L)	1.7	1.4	1.5
Contact Time (h)	1	1	1

Woodward Ave. North:

	Avg SVI, MLSS	High SVI, MLSS	Low SVI, MLSS
Average MLSS (g/L)	2.0	2.6	1.8
SVI (mL/g)	170	240	100
No. Tanks in Series	3	3	3
Flow Split (%)	0/40/60	0/13/87	0/90/10
Recycle Flow (1000 m^3/d)	282	217	329
Last Pass MLSS (g/L)	1.0	1.2	1.0
Contact Time (h)	1.3	1.0	1.1

<u>Step Feed Operation</u>:

Using model of Buhr (1984) with Lotus 123 spreadsheet, the last pass MLSS concentration for step feed operation is calculated to be 1.75 g/L. Using the procedure described above for conventional operation:

Using Equation 3.3,

Initial settling velocity (ISV) = 7.80*Exp[-MLSS*(0.148 + 0.00210*SVI)] =

7.80*Exp[-1.75*(0.148 + 0.00210*169)] = 3.235 m/h

Using Equation 3.1,

Maximum allowable settler overflow rate (SOR) = ISV/SF = 3.235/1.5 = 2.157 m/h

Calculating maximum allowable plant flow rate (Q_{max}),

Q_{max} = SOR*A*24 = 2.157*4737*24 = 245,000 m^3/d

For Port Dalhousie, the capacity under conventional operation ranged from 96% to 544% of the design average flowrate. The substantial variability is due to the variability in the historical MLSS concentration and SVI values. Under step feed the capacity ranged from 301% to 544% of the design average flowrate. The greatest relative increase in capacity occurred with the high MLSS concentration and SVI. At the low MLSS concentration and SVI no increase in capacity was achieved with step feed operation.

For Port Weller, the capacity under conventional operation ranged from 197% to 377% of the design average flowrate. The relatively narrow range resulted from the relatively constant historical MLSS concentration and SVI. Under step feed the capacity ranged from 290% to 421% of the design average flowrate.

For the Main WPCP, the capacity under conventional operation ranged from 74% to 215% of the design average flowrate. Under step feed the capacity ranged from 225% to 289%. The greatest relative increase in plant capacity occurred with the high MLSS concentration and SVI.

For the Humber North WPCP under conventional operation, the capacity ranged from 22% to 287% of the design average flowrate. The very large range resulted from the large MLSS concentration and SVI that occasionally occurred at the plant. Under step feed the capacity ranged from 203% to 372% of the average design flowrate. At the high MLSS concentration and SVI, step feed increased the capacity by approximately 900% in comparison to conventional operation.

For the Woodward Ave. North WPCP under conventional operation, the capacity ranged from 95% to 267% of the design average flowrate. Under step feed, the capacity ranged from 227% to 345% of the average design flowrate.

3.4 General Capacity Estimation Procedure

Several methods could be used to estimate the storm flow capacity of a treatment plant under step feed operation. The most rigorous methods would involve experimental work using the actual plant activated sludge. Other methods may be less accurate but have the advantage of requiring no experimental work. The following are suggested methods of estimating plant hydraulic capacity under step feed operation in order of decreasing complexity:

1. Carry out an experimental program at the plant to actually test the hydraulic capacity of the secondary clarifiers under different flow and solids loading conditions. Apply the Buhr model using the experimentally determined relationship between hydraulic capacity and solids loading to determine the capacity with the optimum step feed scenario.

2. Carry out laboratory tests to measure ISV vs. MLSS concentration for the plant over a period of time. Use the experimentally determined relationship between ISV and MLSS concentration with the Buhr model and Equation 3.1 with an appropriate safety factor to determine the storm flow capacity with the optimum step feed scenario.

3. Use historical plant SVI values or a typical SVI with Equation 3.3 to obtain the relationship between ISV and MLSS concentration. Apply the Buhr model and Equation 3.1 with an appropriate safety factor to determine the storm flow capacity with the optimum step feed scenario.

4. Select a typical SVI value for the plant and a typical step feed MLSS concentration. Apply Equation 3.3 to obtain a typical step feed ISV. Apply Equation 3.1 with an appropriate safety factor to estimate a typical storm flow capacity for the plant with step feed. For this study, a safety factor of 1.5 is recommended for use in Equation 3.1. For general application, it is also suggested that the typical SVI be taken as the average of the five plant average SVI values presented in this study (SVI = 136). It is suggested that an MLSS concentration of 1.25 g/L be used to represent optimum step feed conditions. Using these values, the storm flow capacity of a plant under step feed can be estimated using the following equation:

$$Q_{max} = 72 * A \qquad \text{Equation 3.4}$$

where:

Q_{max} = storm flow capacity under step feed, m^3/d,
A = secondary settler surface area, m^2.

5. The simplest but least rigorous method would be to multiply the plant design flow rate by a typical step feed flow capacity factor. If this method is to be used, it is suggested that the flow capacity factor be the five plant average of the ratio of step feed capacity under average settling conditions to the design flow rate. The step feed flow capacity factor is thus 3.2. Therefore,

$$Q_{max} = 3.2 * Q_{design} \qquad \text{Equation 3.5}$$

where:

Q_{max} = storm flow capacity under step feed, m^3/d,
Q_{design} = plant design flow rate, m^3/d.

4. REQUIRED PLANT MODIFICATIONS

4.1 Introduction

Information concerning the design and operation of each of the five wastewater treatment plants was reviewed by site visits, discussions with plant operators and supervisors, and review of design drawings and documents. For the plants not designed for step feed operation, concepts for modifying the plants for step feed operation were developed. In some cases several alternatives for implementing step feed were possible. In general, the alternatives which appeared, through engineering judgement, to be most easily implemented and least costly were selected. Because of the time constraints of the project, a comprehensive evaluation of alternatives was not carried out. Also due to time constraints, the alternatives and the selected concepts were not discussed with the treatment plant staffs. In the next phase of the project, it is recommended that plant personnel be consulted and have the opportunity to provide input into the development of the step feed concepts to be implemented at their plants.

In general, the cost estimates include only the minimum modifications necessary for implementation of step feed. For example, complete mix aeration tanks were assumed to be baffled to provide only two tanks in series even though greater operational flexibility could be obtained with three or four tanks in series. However, an exception to the general rule of providing for only the minimum requirements was made in the case of control gates and valves. Although it would be possible to use manually operated sluice gates or valves to convert from conventional operation to step feed operation in anticipation of a storm event, it is preferable to employ power actuated gates or valves with remote control from the central control room. Therefore, where power actuated gates and valves were not in place, the costs of adding power actuators and remote controls were included in the cost estimates.

Better control of the plants under storm flow conditions could be achieved through the use of effluent suspended solids and sludge blanket height monitors on the final clarifiers. These instruments could allow operation of the plants closer to their capacity limits. Storm monitoring instrumentation could also be used to anticipate high flow conditions to allow conversion to step feed mode in time to handle the storm flow. These types of instruments are not considered to be essential for the effective use of step feed control and were not included in the plant modification costs.

Based on the selected concepts, preliminary cost estimates for the required plant modifications were made. These estimates should be considered to be feasibility study level cost estimates to be used only for evaluation of storm flow control options. More refined estimates for budget purposes should be based on more detailed conceptual designs. Assistance in estimating costs of mechanical and structural modifications was provided by Bennett Mechanical Installations Ltd.

4.2 Port Dalhousie Treatment Plant

4.2.1 Aeration System

The Port Dalhousie plant uses two separate complete mix aeration tanks operated in

parallel. Each aeration tank has four mechanical draft tube aerators. The primary effluent is discharged with the return activated sludge (RAS) into a collection well located between the two aeration tanks. The mixed primary effluent and RAS is piped from the collection well to two distribution chambers from which it is discharged into the aeration tanks under the draft tube of each aerator. Eight manually operated gates control the flow from the distribution chambers into each aeration section.

4.2.2 Plant Modifications

The Port Dalhousie plant is an example of a relatively difficult case for implementation of step feed. At Port Dalhousie the two aeration tanks are separate complete mix systems. In addition, the RAS is mixed with the primary effluent before being discharged into the aeration tanks. To implement step feed the primary effluent must be separated from the RAS and the aeration tanks must be converted to a tanks in series or multipass system. In addition, it is recommended that power actuators be installed on the eight sluice gates controlling flow from the distribution chambers. The following modifications are required to convert the Port Dalhousie plant for step feed operation (modifications are shown schematically in Figure 12):

1. Construct concrete baffles across the width of each aeration tank to divide the tanks into two approximately equal sections. An overflow weir will be provided on each baffle to allow the mixed liquor to flow from the first to the second section of the tank.

2. Block the eight effluent ports in the first section of each aeration tank.

3. Extend the two RAS lines to discharge into the first section of each aeration tank.

4. Add three manual sluice gates to each of the two RAS lines to control flow to the aeration tanks and the collection well.

5. Add remotely controlled motor actuators to the eight existing sluice gates controlling the influent to the aeration tanks.

4.2.3 Plant Operation

During normal operation, the primary effluent will flow from the collection well into the distribution chambers. The distribution gates will be set such that all or a majority of the primary effluent flows into the first section of each tank. The RAS will flow through the new RAS extensions into the first section of each tank. The mixed liquor will flow from the first tank section into the second section and then into the secondary clarifiers. In anticipation of high flow conditions, the plant will be converted to step feed operation. To convert to step feed, the distribution gates will be set to direct all or part of the primary effluent into the second section of each aeration tank. The most severe step feed action will result when the entire primary effluent is discharged into the second aeration tank section. Distribution of the primary effluent between the first and second tank sections will result in a less severe step feed action. The more severe the step feed action, the greater the reduction in solids loading to the secondary clarifiers will be.

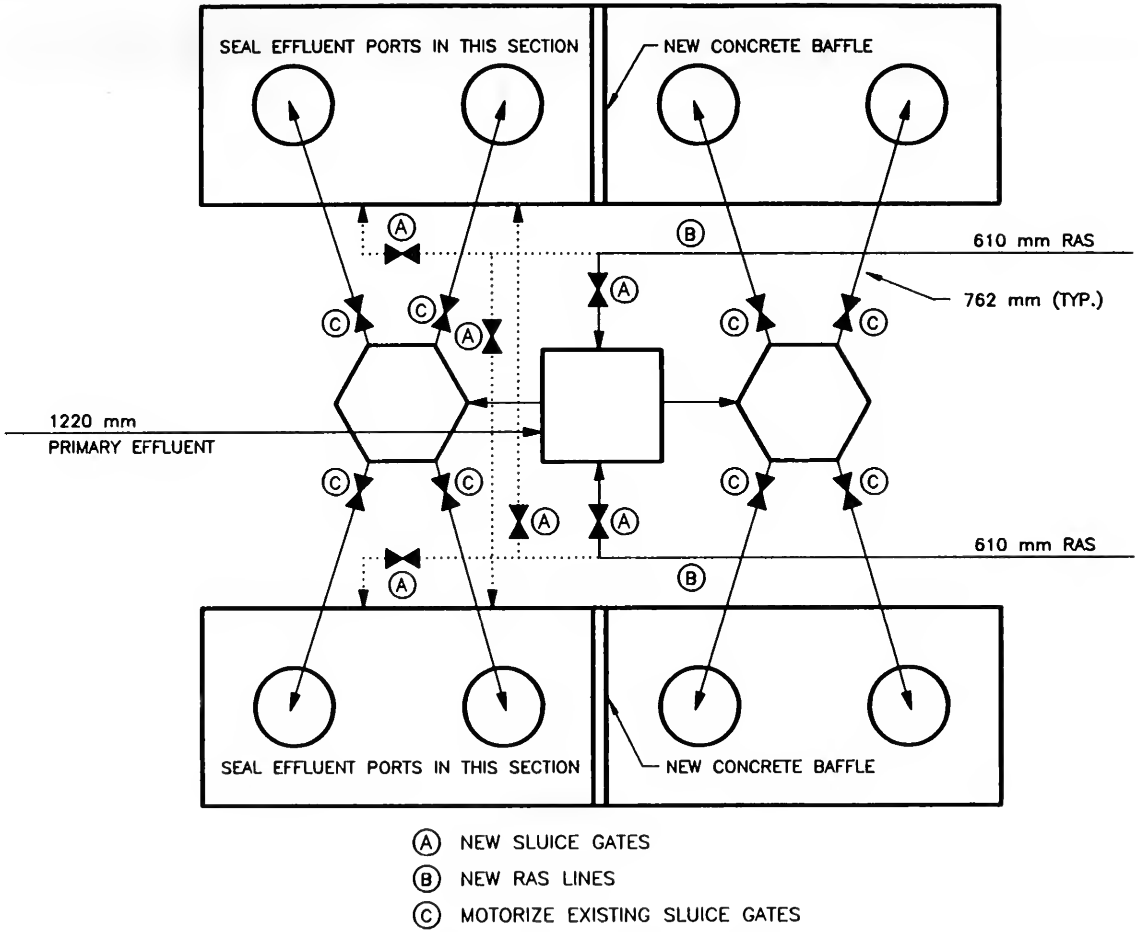

Figure 12 – Port Dalhousie Plant Step Feed Modifications

4.2.4 Cost Estimate

Table 4.1 gives a breakdown of the estimated costs for implementation of the modifications to convert the Port Dalhousie treatment plant to allow step feed operation. The total cost for the modifications is estimated to be $410,000.

Table 4.1- Cost Estimate for Port Dalhousie Treatment Plant

Item	Estimated Cost ($)
Sluice Gate Operators (8)	21,000
Operator Installation	12,000
Sluice Gates	42,000
Miscellaneous Metal	30,000
Civil Works	83,000
Excavation, Bedding, Backfill	10,000
Mechanical Works	44,000
Electrical	54,000
Sub-Total	296,000
Contractor's Mark-up	45,000
Sub-Total	341,000
Contingency	69,000
Total	410,000

4.3 Port Weller Treatment Plant

4.3.1 Aeration System

The aeration system at the Port Weller plant is similar to the system at the Port Dalhousie plant. Two separate complete mix aeration tanks are operated in parallel. Each aeration tank has four mechanical draft tube aerators. The primary effluent is discharged with the RAS into a collection well located between the two aeration tanks. The mixed

with the RAS into a collection well located between the two aeration tanks. The mixed primary effluent and RAS is piped from the collection well to two distribution chambers from which it is discharged into the aeration tanks under the draft tube of each aerator. Eight manually operated gates control the flow from the distribution chambers into each aeration section. However the Port Weller plant has the flexibility to discharge the RAS directly to the aeration tanks without mixing with the primary effluent. In this plant the RAS line extends to the head end of each aeration tank. Manually operated sluice gates control the flow of RAS into the aeration tanks. Another manual sluice gate controls the flow of RAS into the collection well. Currently the plant is operated with all the RAS discharging into the collection well.

4.3.2 Plant Modifications

The Port Weller plant is an example of a moderate cost case for implementation of step feed. At Port Weller the two aeration tanks are separate complete mix systems. However, the RAS can be separated from the primary effluent before being discharged into the aeration tanks. To implement step feed the aeration tanks must be converted to a tanks in series or multipass system. Although not essential, it is also recommended that the manual gates be power actuated. The following modifications are required to convert the Port Weller plant for step feed operation (modifications are shown schematically in Figure 13):

1. Construct concrete baffles across the width of each aeration tank to divide the tanks into two approximately equal sections. An overflow weir will be provided on each baffle to allow the mixed liquor to flow from the first to the second section of the tank.

2. Block the eight effluent ports in the first section of each aeration tank.

3. Add remotely controlled motor actuators to the eight existing sluice gates controlling the influent to the aeration tanks.

4.3.3 Plant Operation

During normal operation, the primary effluent will flow from the collection well into the distribution chambers. The distribution gates will be set such that all or a majority of the primary effluent flows into the first section of each tank. The RAS will flow into the first section of each tank. The mixed liquor will flow from the first tank section into the second section and then into the secondary clarifiers. In anticipation of high flow conditions, the plant will be converted to step feed operation. To convert to step feed, the distribution gates will be set to direct all or part of the primary effluent into the second section of each aeration tank. The most severe step feed action will result when the entire primary effluent is discharged into the second aeration tank section. Distribution of the primary effluent between the first and second tank sections will result in a less severe step feed action. The more severe the step feed action, the greater the reduction in solids loading to the secondary clarifiers will be.

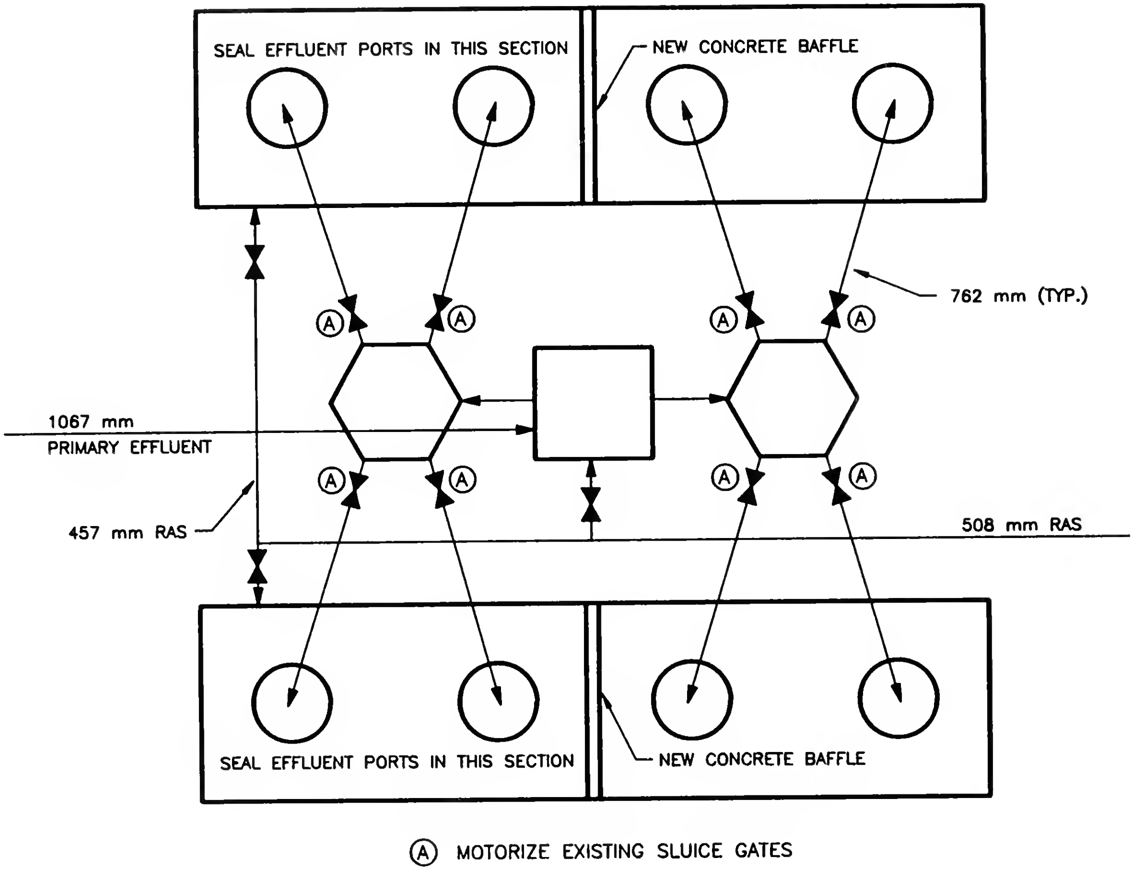

Figure 13 – Port Weller Plant Step Feed Modifications

4.3.4 Cost Estimate

Table 4.2 gives a breakdown of the estimated costs for implementation of the modifications to convert the Port Weller treatment plant to allow step feed operation. The total cost for the modifications is estimated to be $240,000.

Table 4.2- Cost Estimate for Port Weller Treatment Plant

Item	Estimated Cost ($)
Sluice Gate Operators (8)	21,000
Operator Installation	12,000
Miscellaneous Metal	30,000
Civil Works	57,000
Electrical	54,000
Sub-Total	174,000
Contractor's Mark-up	26,000
Sub-Total	200,000
Contingency	40,000
Total	240,000

4.4 Humber North Treatment Plant

4.4.1 Aeration System

The Humber treatment facility is made up of two essentially separate plants, the South Plant and the North Plant. The two plants are not entirely separate in that cross connections exist between the plants to allow some flexibility in operations. The South Plant has five 3-pass aeration tanks each designed for step feed operation. The aeration tanks employ fine bubble diffused air systems. The South Plant routinely operates in a step feed mode, splitting the primary effluent between the three passes of each tank. During normal operation, the gates to the first and third passes of each tank are open approximately 25% and the gates to the second pass of each tank are open approximately 50%. During storm flow conditions all gates are open 100%.

In the North Plant three separate complete mix aeration tanks are operated in parallel. Each aeration tank employs a fine bubble diffused air system. The primary effluent is discharged with the RAS into influent troughs located at the head end of each of the three aeration tanks. In each tank the mixed primary effluent and RAS flows from the influent trough to two elevated channels which discharge through overflow weirs into the aeration tank approximately one fourth of the distance along the length of the tank.

4.4.2 Plant Modifications

The Humber South Plant requires no modifications for step feed operation. The Humber North Plant is an example of a relatively difficult case for conversion to step feed operation. At the North Plant the three aeration tanks are separate complete mix systems, and the RAS and primary effluent are mixed before being discharged into the aeration tanks. To implement step feed, the aeration tanks must be converted to a tanks in series or multipass system and the RAS and primary effluent must be separated. The following modifications are required to convert the Humber North Plant for step feed operation (modifications are shown schematically in Figure 14):

1. Construct concrete baffles across the width of each aeration tank to divide the tanks into two sections. To avoid interference with the air system the baffles can be placed approximately 3/4 down the length of the tanks. An overflow weir will be provided on each baffle to allow the mixed liquor to flow from the first to the second section of the tank. A drain opening will also be placed at the bottom of each baffle.

2. Extend the RAS pipe in each tank through the wall of the influent trough to discharge directly into the head end of the aeration tank.

3. In each aeration tank, install a 610 mm diameter steel pipe through the wall of the influent trough and extending along the length of the tank into the second chamber.

4. In each tank, install a remotely controlled, motor actuated sluice gate to the new influent pipe at the influent trough wall to control the flow of primary effluent into the second chamber of the tank.

4.4.3 Plant Operation

During normal operation, the new sluice gates will be fully or partially closed. All or most of the primary effluent will flow into the influent troughs and out through the existing channels into the first sections of the aeration tanks. The RAS will flow directly into the first section of each tank through the new RAS pipe extensions. The mixed liquor will flow from the first tank section into the second section and then into the secondary clarifiers. In anticipation of high flow conditions, the plant will be converted to step feed operation. To convert to step feed, the new sluice gates will be opened to direct a greater portion of the primary effluent into the second section of each aeration tank. The most severe step feed action will result when the entire primary effluent is discharged into the second aeration tank section. Distribution of the primary effluent between the first and second tank sections will result in a less severe step feed action. The more severe the step feed action, the greater the reduction in solids loading to the secondary clarifiers will be.

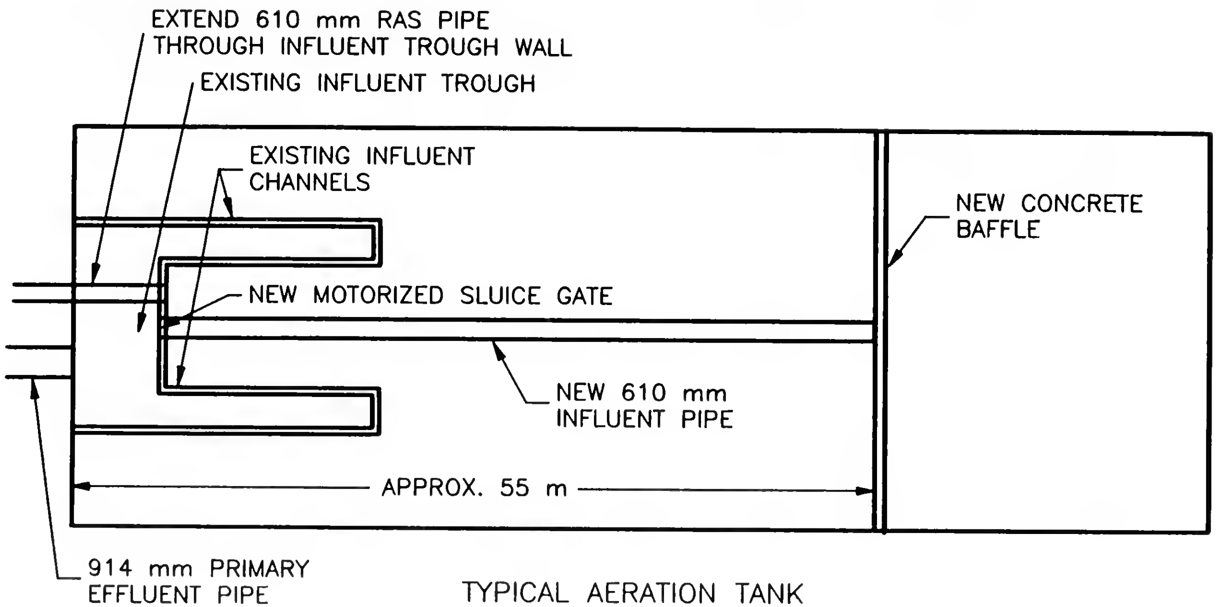

Figure 14 – Humber North Plant Step Feed Modifications

4.4.4 Cost Estimate

Table 4.3 gives a breakdown of the estimated costs for implementation of the modifications to convert the Humber North Plant to allow step feed operation. The total cost for the modifications is estimated to be $610,000.

Table 4.3- Cost Estimate for Humber North Treatment Plant

Item	Estimated Cost ($)
Sluice Gates with Operators (3)	30,000
Pipe Material	120,000
Pipe Installation	36,000
Civil Works	235,000
Electrical	25,000
Sub-Total	446,000
Contractor's Mark-up	67,000
Sub-Total	513,000
Contingency	97,000
Total	610,000

4.5 Toronto Main Treatment Plant

4.5.1 Aeration System

The Toronto Main Treatment Plant has nine 4-pass aeration tanks each designed for step feed operation. The aeration tanks employ fine bubble diffused air systems. The Main Plant routinely operates in a step feed mode, splitting the primary effluent between the four passes of each tank. During normal operation, the locally controlled, motor operated gates to the first through the fourth pass of each tank are open approximately 100%, 75%, 50%, and 25%, respectively. If the flow splits are proportional to the gate openings, this results in a flow split to the first through the fourth pass of 40%, 30%, 20%, and 10%, respectively. Flow measurements are not available on the aeration tanks so the flow splits between tanks and between passes in individual tanks are not known. During storm events the severity of the step feed action could be increased by setting the gates to distribute the influent flow

the step feed action could be increased by setting the gates to distribute the influent flow further downstream in the aeration tanks. This would further reduce solids loading to the final clarifiers.

Since the plant is designed for and operated in step feed mode, no plant modifications are required. However, it is desirable to add remote controls to some of the flow control gates. To simplify the control operations, only two gates in each tank might be remotely controlled. Those gates could be adjusted to increase or decrease the severity of the step feed action depending on the flow to the plant. The other two gates would be maintained in preset positions.

4.5.2 Plant Modifications

The Toronto Main Treatment Plant is an example of a relatively low cost case for implementation of step feed. The plant is currently designed for and operated in a step feed mode. The gates controlling the step feed flow distribution are already motor actuated but not remotely controlled. Although not essential, for the purposes of this study it will be assumed that two gates in each aeration tank be remotely controlled from the central control room.

4.5.3 Plant Operation

During normal operation, the primary effluent will be distributed to the four passes of each aeration tank through the motor operated influent gates. In anticipation of high flow conditions, a more severe step feed action may be taken by adjusting the remotely controlled gates to distribute more influent flow to downstream passes of the aeration tanks.

4.5.4 Cost Estimate

Table 4.4 gives a breakdown of the estimated costs for implementation of the modifications to convert the Toronto Main Plant for step feed operation. The total cost for the modifications is estimated to be $110,000.

4.6 Hamilton Woodward Avenue North Treatment Plant

4.6.1 Aeration System

The Woodward Avenue treatment plant uses two separate secondary treatment systems known as the North Plant and the South Plant. Each of the two systems contains aeration tanks and secondary clarifiers. There are no cross connections between the two secondary treatment systems. Both systems are fed from the same pretreatment and primary treatment systems. The North Plant was originally designed to accommodate step feed operation, however the plant is not currently operated in a step feed mode. The newer South Plant was not designed for step feed operation and utilizes complete mix aeration tanks with mixing of the primary effluent and RAS before discharge into the aeration tanks. In this

phase of the study only the North Plant will be considered.

Table 4.4- Cost Estimate for Toronto Main Treatment Plant

Item	Estimated Cost ($)
Add remote controls to influent gates (18)	80,000
Contractor's Mark-up	12,000
Sub-Total	92,000
Contingency	18,000
Total	110,000

The aeration system at the Woodward Avenue North Plant consists of eight aeration tanks operated in parallel. Each tank has six mechanical surface aerators. Partial baffles between the aerators divide each tank into six sections. During normal operation, the primary effluent is discharged into the head end of each tank through two manually operated sluice gates. The RAS is separately discharged into the head end of each tank.

A channel runs between each pair of aeration tanks along the length of the tanks. Eight influent ports with manually adjustable weirs are positioned along the length of each tank. A motor operated sluice gate controls the flow of primary effluent into each of the four channels. To operate in step feed mode, the main influent gates in each tank must be closed and the channel gates opened. Primary effluent will then be directed to the channels and into the aeration tanks through the influent ports along the length of the tanks.

4.6.2 Plant Modifications

The Hamilton Woodward Avenue North Plant is an example of a moderate cost case for implementation of step feed. At the North Plant the eight aeration tanks are baffled to provide a tanks in series hydraulic regime. In addition, the RAS is separate from the primary effluent. It is recommended that one of the manually operated influent gates on each tank be power actuated. The other influent gate on each tank should be normally closed. The eight motor operated influent gates and the four motor operated channel inlet gates should be remotely controlled from the central control room. Since the eight step feed influent ports cannot be closed completely, it is recommended that six ports in each tank be sealed so that the influent can be directed to only two ports. One of the open ports will discharge into the fourth section and the other into the fifth section. The adjustable weirs will allow for control of the flow split between the two ports to adjust the severity of the step

feed action. The following modifications are required to convert the Woodward Avenue North Plant for step feed operation (modifications are shown schematically in Figure 15):

1. Seal six of the step feed influent ports in each aeration tank.

2. Add remotely controlled motor actuators to the eight existing sluice gates controlling the influent to the aeration tanks.

3. Add remote controls to the four step feed channel gates.

4.6.3 Plant Operation

During normal operation, the primary effluent will flow into the head end of each aeration tank through the motor operated influent sluice gate on each tank. The second influent gate on each tank will remain closed. The RAS will be discharged into the head end of each aeration tank through the RAS pipe. In anticipation of high flow conditions, the plant will be converted to step feed operation. To convert to step feed, the influent gates on each tank will be closed. The four channel inlet gates will be opened directing the primary effluent into the channel and from the channel into the fourth or fifth section of each aeration tank. The most severe step feed action will result when the entire primary effluent is discharged into the fifth aeration tank section. Less severe step feed actions can be obtained by distributing more of the primary effluent to the fourth tank section. The more severe the step feed action, the greater the reduction in solids loading to the secondary clarifiers will be.

4.6.4 Cost Estimate

Table 4.5 gives a breakdown of the estimated costs for implementation of the modifications to convert the Hamilton Woodward Avenue North Plant for step feed operation. The total cost for the modifications is estimated to be $270,000.

4.7 Cost Summary

Table 4.6 gives a summary of the cost estimates for modifying the five plants for step feed operation.

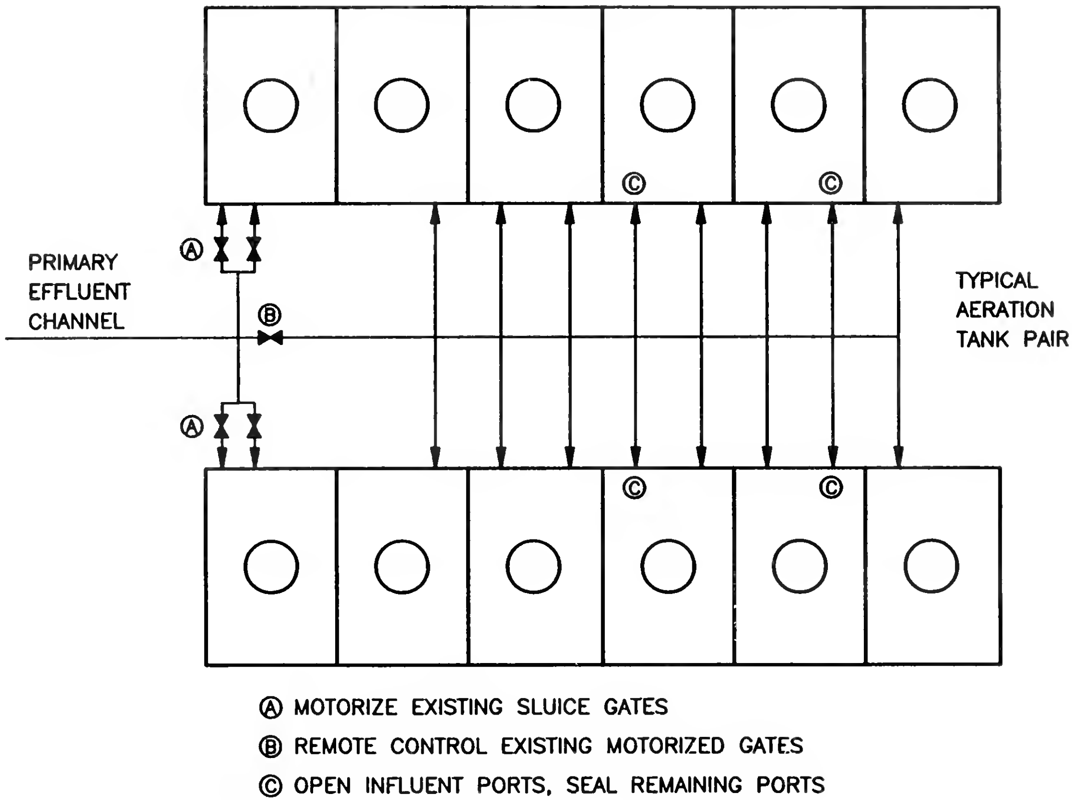

Figure 15 – Woodward Avenue North Plant Step Feed Modifications

Table 4.5- Cost Estimate for Woodward Ave. North Treatment Plant

Item	Estimated Cost ($)
Sluice Gate Operators (8)	48,000
Operator Installation	12,000
Seal Influent Ports (48)	53,000
Electrical	80,000
Sub-Total	193,000
Contractor's Mark-up	29,000
Sub-Total	222,000
Contingency	48,000
Total	270,000

Table 4.6- Summary of Estimated Costs for Plant Modifications

Plant	Cost ($)	Modification Category
Port Dalhousie	410,000	Substantial
Humber North	610,000	Substantial
Port Weller	240,000	Moderate
Woodward Avenue North	270,000	Moderate
Toronto Main	110,000	Minimal

5. GENERAL COST ESTIMATION PROCEDURE

5.1 Unit Costs

This section presents the plant step feed modification cost estimates on a unit cost basis. The unit costs are intended to be used as an aid for estimating the cost impact of implementing the Ontario Ministry of the Environment policy on combined sewer overflows. Table 5.1 gives the modification cost for each plant per unit of design average flowrate. Where available, the design flow was obtained from the plant Certificate of Approval.

The five plants studied can be divided into three general categories based on the complexity of the modifications required to implement step feed operation. The Port Dalhousie and Humber North plants require substantial modifications. The Port Weller and Woodward Avenue North plants require moderate modifications. The Toronto Main plant requires minimal modifications. The modification categories for each plant are also presented in Table 5.1.

5.2 Guidelines for Application of Unit Cost Data to Other Plants

To estimate the cost of converting other plants to step feed operation, the complexity of the modifications required must first be assessed. Table 5.2 presents suggested guidelines to determine the level of complexity of the modifications required for conversion to step feed for use in applying the unit cost data to other plants. When the level of complexity of the required modifications has been determined, the modification cost range may be estimated by multiplying the appropriate unit costs by the plant design average flowrate. The unit costs developed in this study are based on a very limited investigation and therefore should only be used for order of magnitude cost estimates.

Table 5.1- Unit Step Feed Conversion Cost Estimates

Plant	Design Flow (10^3 m^3/d)	Cost ($)	Unit Cost ($/$10^3$ m^3/d)	Modification Category
Port Dalhousie	61	410,000	6700	Substantial
Humber North	177	610,000	3400	Substantial
Port Weller	56	240,000	4300	Moderate
Woodward Ave. North	273	270,000	1000	Moderate
Toronto Main	818	110,000	130	Minimal

Table 5.2- Suggested Guidelines for Determining Step Feed Modification Complexity

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Substantial Modifications Required

Unit Cost Range: 3400 - 6700 [$/($10^3$ m^3/d)]

1. Primary effluent and RAS are mixed prior to discharge into aeration tanks. Primary effluent and RAS lines must be rerouted to enter aeration tanks separately, or

2. Aeration tanks require conversion from complete mix to tanks in series mixing regime, and primary effluent requires rerouting to discharge downstream in aeration tanks.

Moderate Modifications Required

Unit Cost Range: 1000 - 4300 [$/($10^3$ m^3/d)]

1. Primary effluent and RAS are discharged separately into aeration tanks.

2. Aeration tanks require conversion from complete mix to tanks in series mixing regime, or primary effluent requires rerouting to discharge downstream in aeration tanks.

Minimal Modifications Required

Unit Cost Range: 0 - 130 [$/($10^3$ m^3/d)]

1. Plant requires only minimal modifications for step feed operation, e.g., addition of power actuators and/or remote control to valves or gates.

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